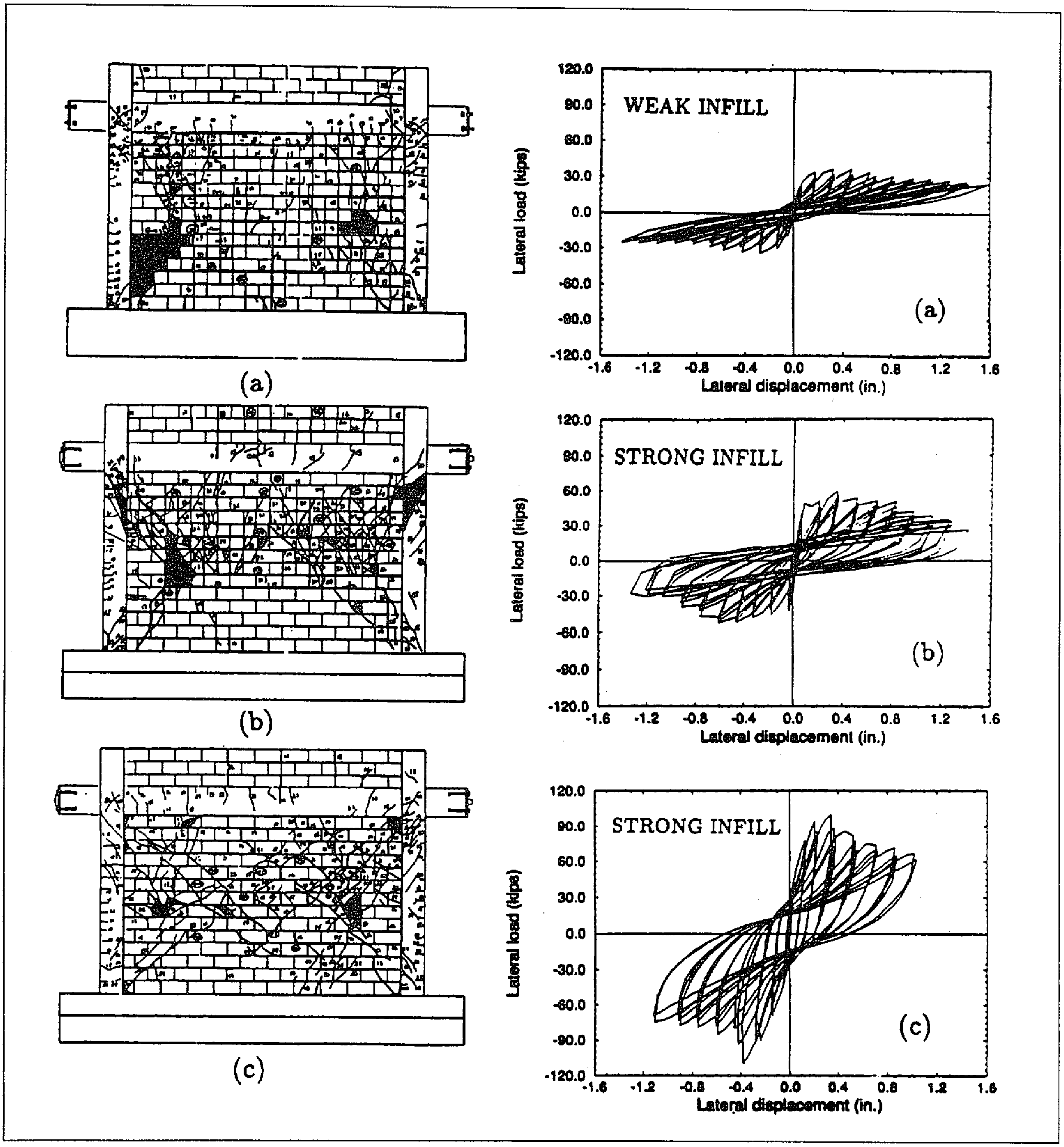
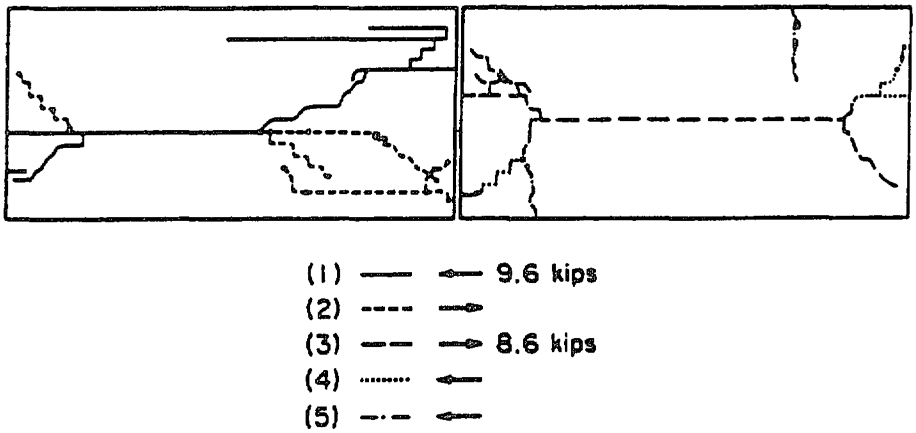
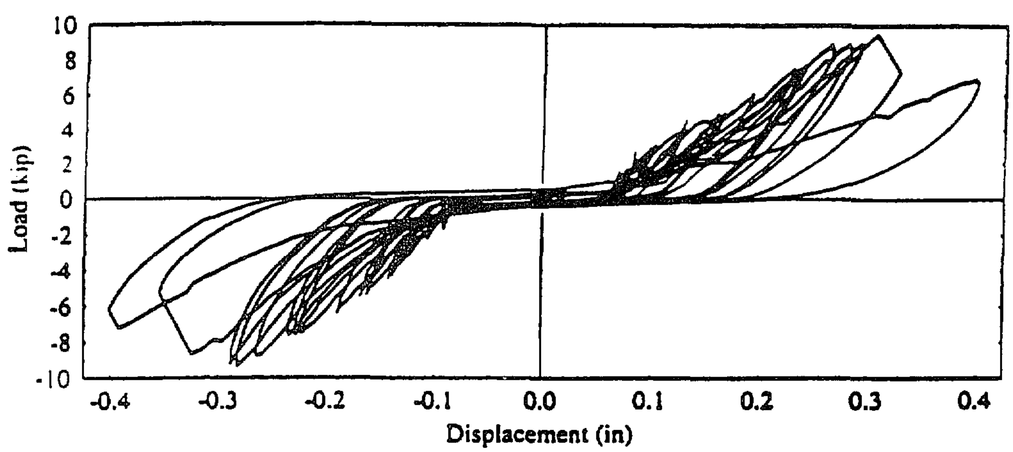


Figure 8-1 *Ductile reinforced concrete frames with concrete masonry infills tested by Mehrabi et al. (1996). (The weak and strong infills were ungrouted and grouted, respectively). (a) Specimen 4, (b) Specimen 5, (c) Specimen 7; h/L aspect ratio = 0.67. Note 1 in. = 1.65% interstory drift*



Cracking sequence of the second two-bay infilled frame.



Load–displacement relationship of the second two-bay infilled frame.

Figure 8-2 *Bed-joint sliding of a two-bay steel frame-block infill. Model study by Gergely et al. (1994).*

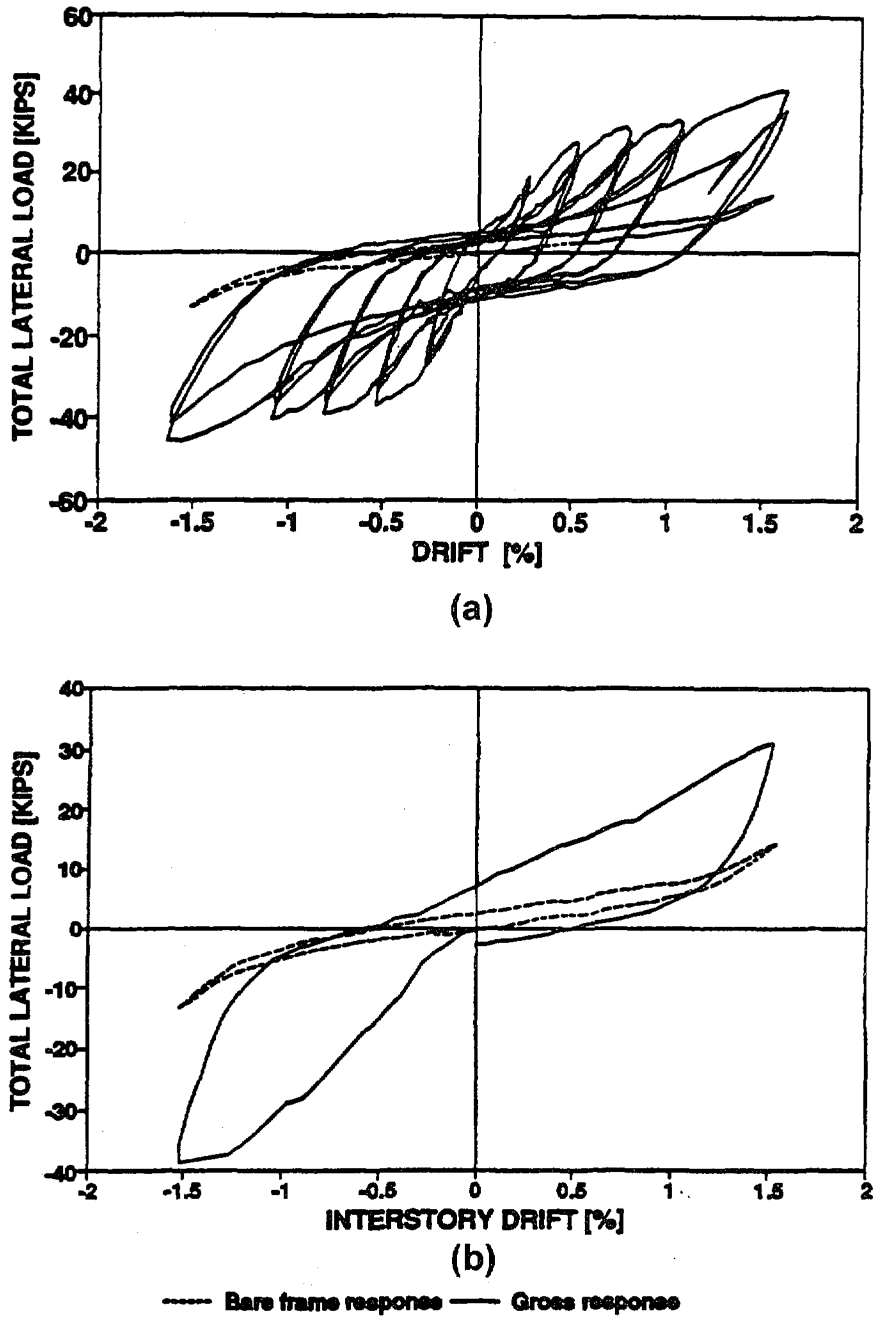
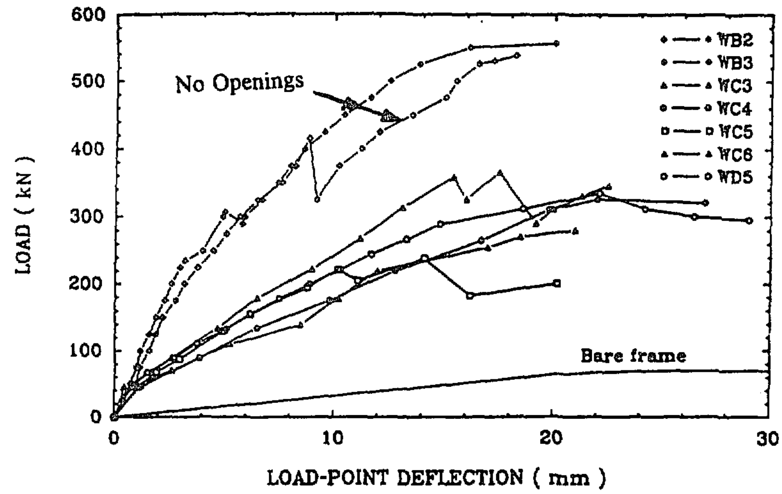
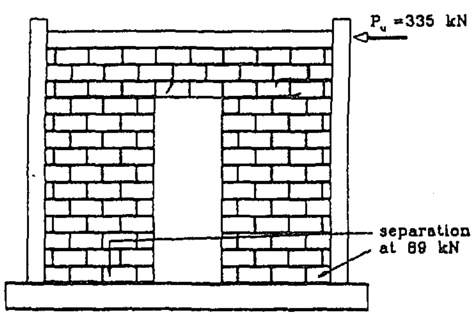


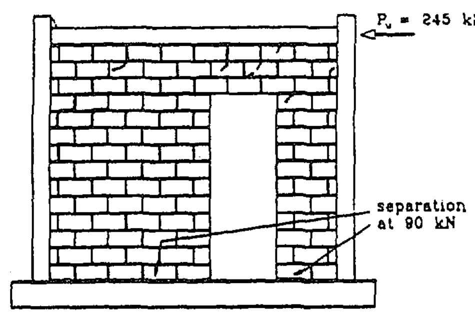
Figure 8-3 *Specimen tested by Mander et al. (1993a). Steel frame-clay brick masonry infill. Top and seat angles semi-rigid connections used to connect beams to columns.*
(a) Original specimen.
(b) Specimen repaired with 1/2-inch ferrocement overlay and retested.



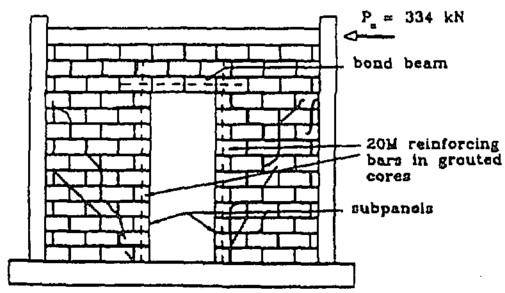
(a) . Effects of openings.



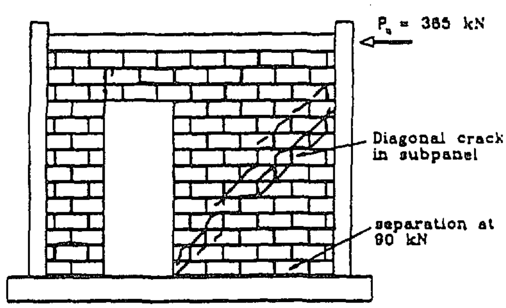
(b) Crack patterns for a specimen (WC4) with central doorway opening.



(c) Crack patterns for a specimen (WC5) with opening offsets towards opening.



(d) Crack patterns for a specimen (WD5) with reinforced central doorway opening.



(e) Crack patterns for a specimen (WC6) with opening offsets away from load.

Figure 8-4 ***Effect of openings on the monotonic lateral-load performance of steel frame-masonry infill tested by Dawe and Seah (1988).***

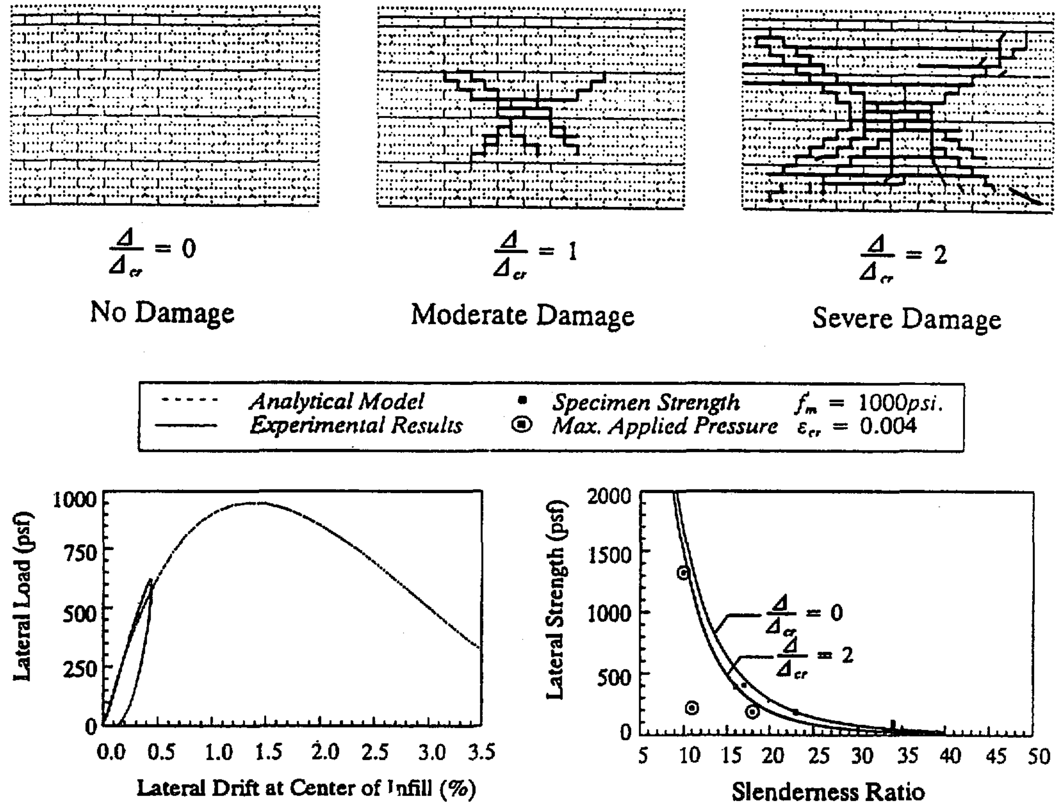


Figure 8-5 ***Out-of-plane behavior of infilled masonry walls showing crack patterns and out-of-plane lateral load vs. lateral displacement of an air bag test. (From Angel and Abrams, 1994).***

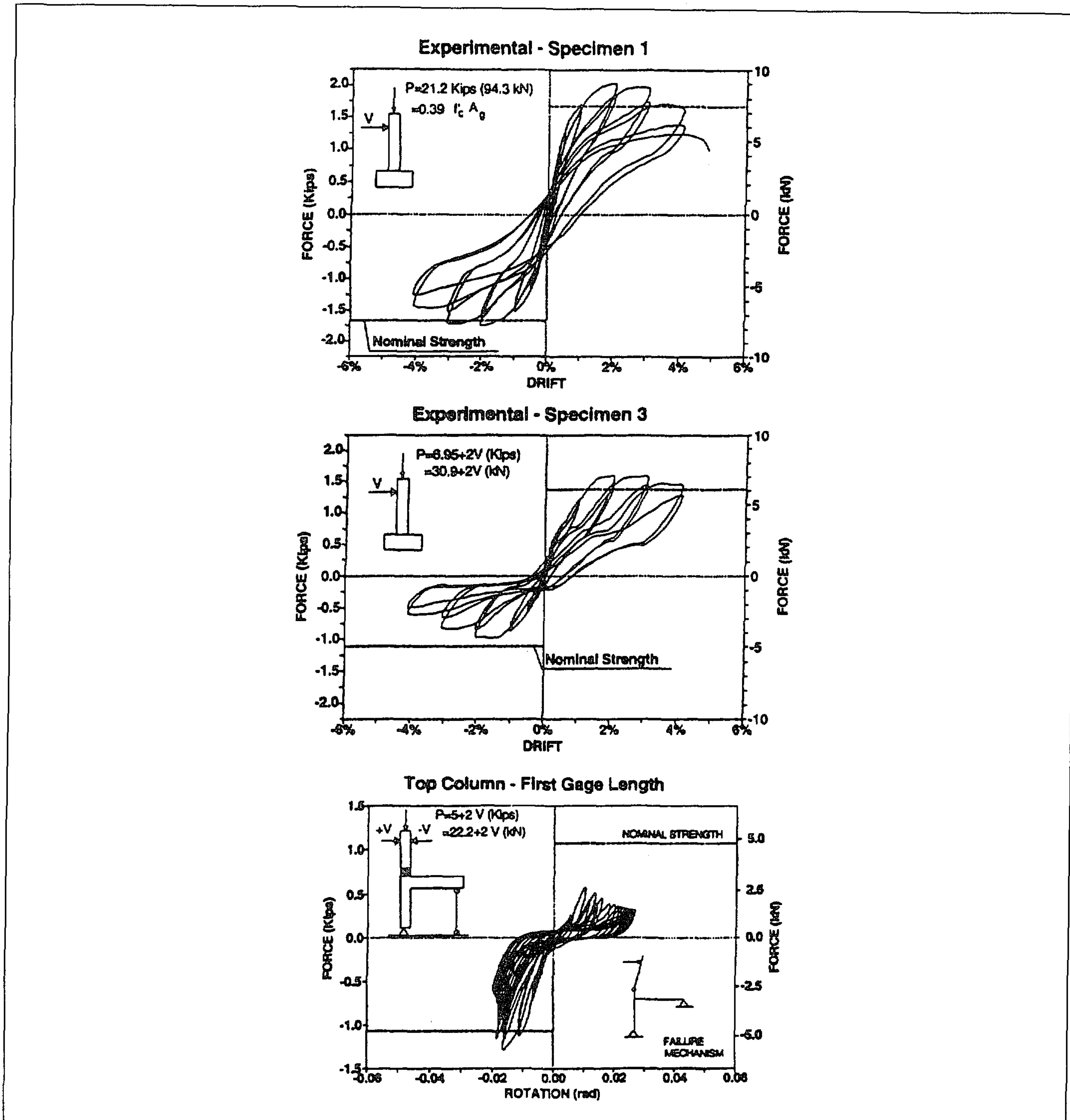


Figure 8-6 *Experiments conducted by Aycardi et al. (1994), showing the performance of nonductile frame members with lap splices at the base of the column.*
Specimen 1: Column with a moderately high level of axial load.
Specimen 3: Column with a lower level, variable, axial load.
Slab-beam-column subassemblage: Tested with variable axial load.
(Note: Deterioration was due to cyclic loading action, the weak zone being the beam rather than the column).

b. Panels With Openings

Infilled panels with openings are best viewed as assemblies of subcomponents of the appropriate material. The behavior modes for the subcomponents are discussed in the other material sections (Chapter 5 for concrete, Chapter 6 for reinforced masonry, and Chapter 7 for URM). These subcomponents interact with the surrounding frame and can alter the frame response. Principal types of interaction can that can occur are strong columns and strong piers inducing shear failure in the beams, strong spandrel components reducing the ductility by causing short-column effects, and by the infill inducing tension yielding or bar splice failures in the column. A discussion of the frame component behavior modes is given below in Section 8.2.3c.

c. Frame components

Table 8-3 and Table 8-4 present the principal behavior modes for steel and concrete frames, respectively, possessing infills. An explanation of these behavior modes follows.

i. Flexural Yielding in Steel: Flexural yielding of the frame is primarily associated with steel frames. When large lateral loads are imposed on moment frames, flexural yielding that leads to plastic hinging is to be expected adjacent to rigid connection. For infilled frames, this generally occurs at the base of fixed-base columns. This is generally evidenced by cracking of paint (if any) and buckling of compression flanges. As the rotational capacity of flexural plastic hinges in steel members is high, and unlikely to be attained in infilled-frame systems because the infills limit the interstory drifts, damage from this behavior mode is mostly cosmetic and generally not serious.

ii. Shear Yielding in Steel: When corner crushing occurs in a strong infill, the diagonal compression strut moves downward into the column providing a large shear force at the end-region of the column. For thin webbed steel members (non-compact sections) that are not confined by concrete, this may lead to web buckling and large localized shear deformations in the frame members. However, shear yielding in steel is ductile, and damage arising from this behavior mode is not serious.

iii. Bolted or Riveted Connection Failure: Steel-frame systems with infill panels generally have bolted or riveted semi-rigid, beam-to-column connections, and the connection is usually encased in concrete. Under these conditions the joint will behave as a rigid joint until the concrete breaks. Under large lateral loads the concrete can fail, or, when there is no confinement, yielding of the connection may occur. Such behavior is not serious, because the ductility capacity of semi-rigid connections is considerable (Mander et al., 1994 and 1995). However, in the presence of infill panels, the large diagonal strut force puts the connection under considerable axial tension. This may cause prying in the connection angles, giving the appearance of serious damage. Nevertheless, the ductility capability of these connections is still considerable and they are capable of sustaining many cycles of loading before a low-cycle fatigue failure, by which time the infill panel itself will have inevitably failed (Mander et al., 1993a).

iv. Flexural Yielding in Reinforced Concrete Frame Components: This behavior mode is expected to occur in reinforced concrete frames with infill when the interstory drift ratios exceed about 0.005. Flexural yielding behavior occurs where moments are greatest—that is, at the ends of beam and column members. In order for a complete side-sway mechanism to form in a structural concrete frame, flexural yielding and plastic hinging must also occur at the base of the columns (ground-floor level). Flexural yielding behavior in reinforced concrete beam and column components is characterized by tensile cracking in the cover concrete (transverse to the axis of the member), coupled with some compression crushing in the cover concrete on the opposite face. High bending moments in frames are also generally associated with high shear forces. Shear demand, when coupled with bending moment, produces diagonal cracking. When flexural plastic-hinge rotations become substantial (note that this is unlikely for infilled frames as the infills substantially limit the amplitudes of interstory drifts), considerable crushing and loss of cover concrete is evident, often leaving the longitudinal and transverse reinforcement exposed. If such severe damage is evident in an infilled-frame system, it is likely to occur in the lowest story, where high story shear demand has caused the infill panel to fail, leading to subsequent, high, interstory drifts (see, for example, Klingner and Bertero, 1978).

v. Lap-Splice Slip in Concrete: Most older infilled-frame structures have not been specifically designed for earthquake resistance, and the frames possess nonductile details. This can lead to column lap splices occurring in potential plastic-hinge zones. For the lap-splice detail deficiency, the connection will undergo flexural yielding at large lateral drifts. Cracking associated with incipient hinging tends to show a combination of transverse (flexure) cracks along with longitudinal (splitting) cracks that run parallel to the longitudinal column reinforcement. These longitudinal cracks signal that the bars in the lap-splice zone have begun to slip. If the interstory drifts are substantial and the cyclic loading pronounced, then the bond within the lap-splice zone is destroyed. Moment transfer in the lap-splice zone becomes limited when spalling of the cover concrete is apparent. It should be noted, however, that this behavior mode does not necessarily lead to an unsafe situation, for the lack of moment capacity finally leads to a pinned connection, capable of transferring axial load and shear. For some experimental results on nonductile concrete columns with lap splices at the base of columns, see Aycardi et al. (1992, 1994).

vi. Column Tension Yielding: When the effect of the infill is substantial, the frame will behave more like a braced frame than moment frame, resulting in the columns resisting the lateral forces and overturning forces in tension and compression. For many older buildings the columns are lightly reinforced and may have more compression capacity than tension capacity, resulting in a tension yielding condition. This is a ductile mode, allowing larger displacements without causing an abrupt failure. The limiting deformation in this mode is the deformation capacity of the infill.

vii. Concrete Shear Failure: Infilled frames that possess strong infill panels generate large shear forces in the infill panels when under lateral side-sway. These shear forces must be transferred from the panel into the frame. If the infill panel is damaged in the corners as a result of corner crushing, then the diagonal strut tends to move away from the panel corner. The high transverse forces from the diagonal strut then enter the frame some distance (typically about one member depth) away from the beam/column connection. This provides a very high shear demand over a short column (or sometimes beam) length. Damage to the frame members is indicated by large diagonal X-cracks and spalling of the cover concrete. Under very severe cases of damage (weak frame / strong infill), complete loss of cover concrete and bulging of the core may be expected. If this occurs in a column, it is a serious form of damage, because the ability of the column to transfer axial loads may be seriously impaired. Therefore, it is not surprising that the ductility capability of such shear-critical elements is low.

viii. Concrete Joint Failure: Beam/column joints are subjected to high shear forces when under lateral loading. These shear forces can be amplified when infills are present. For concrete frames with nonductile reinforcing steel details, there is generally a deficiency or complete absence of transverse reinforcement within the beam/column joint core. Therefore, the shear-strength capacity is inevitably less than the demand imposed, even at moderate interstory drifts. Consequently, this highly likely behavior mode leads to large X-cracks in the beam-column joint region. Under cyclic loading, the cover concrete spalls, the joint concrete bulges, and the longitudinal column reinforcement tends to buckle. Such behavior tends to keep the adjacent beam and column plastic-hinge regions from being severely damaged. However, the ability of the frame system to carry axial loads through the damaged joints is suspect, especially if the behavior mode is associated with an adjacent infill panel that is also near failure.

Table 8-3 Behavior Modes For Infilled Steel-Frame Components

Behavior Mode	Description, and Likelihood of Occurrence	Ductility	Figure	Paragraph (see Section 8.2.3c):
Flexural yielding	Hinges form at base of columns, and can occur adjacent to the beam/column joint if the members are weaker than the connection.	High	Similar to 8-1(a)	i
Shear yielding	Unlikely, except when infill causes short-column effect.			ii
Bolted or riveted connection failure	Likely.	High	8-3	iii

Table 8-4 Behavior Modes For Infilled Concrete-Frame Components

Behavior Mode	Description, and Likelihood of Occurrence	Ductility	Figure	Paragraph (see Section 8.2.3c):
Flexural yielding	Should always occur at ground-floor level. Probably will also occur adjacent to beam/column joint.	High	8-1c	iv
Lap-splice slip	Probable, will normally occur at ground-floor level.	Moderate-to-low	8-6	v
Column tension yielding	Lightly reinforced columns with strong infill	Moderate-to-high		vi
Shear failure	Probable in nonductile frames. Likely at partial-height infills.	Low	8-1b	vii
Joint failure	Probable with nonductile detailing.	Low	8-1a	viii

8.3 Infilled Frame Evaluation Procedures

8.3.1 Solid Infilled-Panel Components

This subsection gives equations for quantifying the stiffness, strength and deformation capacity of infilled panels. Note that Young's modulus and strength values for the infill panel are given in terms of masonry materials. For reinforced concrete infills, make the following substitutions: $E_c = 57000\sqrt{f'_{ce}}$ for E_m and f'_{ce} for f'_{me}.

a. Stiffness

The effective width(s) of a diagonal compression strut that can be used to assess the stiffness and strength of an infill panel is initially calculated using the recommendations given in FEMA 273. The provisions are based on the early work of Mainstone (1971) and Mainstone and Weeks (1970) and are restated below for the convenience of the user.

The equivalent strut is represented by the actual infill thickness that is in contact with the frame (t_{inf}) and the diagonal length (r_{inf}) and an equivalent width, a, given by:

$$a = 0.175(\lambda_1 h_{col})^{-0.4} r_{inf} \qquad (8\text{-}1)$$

where

$$\lambda_1 = \left[\frac{E_{me} t_{inf} \sin 2\theta}{4 E_{fe} I_{col} h_{inf}}\right]^{\frac{1}{4}} \qquad (8\text{-}2)$$

and

h_{col} = column height between centerlines of beams, in.

h_{inf} = height of infill panel, in.

E_{fe} = expected modulus of elasticity of frame material, psi.

E_{me} = expected modulus of elasticity of infill material, psi.

I_{col} = moment of inertial of column, in^4.

r_{inf} = diagonal length of infill panel, in.

t_{inf} = thickness of infill panel and equivalent strut, in.

θ = angle whose tangent is the infill height-to-length aspect ratio, radians

$$\theta = \tan^{-1}\left(\frac{h_{inf}}{L_{inf}}\right) \qquad (8\text{-}3)$$

where

L_{inf} = length of infill panel, in.

Only the masonry wythes in full contact with the frame elements need to be considered when computing in-plane stiffness, unless positive anchorage capable of transmitting in-plane forces from frame members to all masonry wythes is provided on all sides of the walls.

b. Strength

The strength capacity of an infill panel is a complex phenomenon. It is important to analyze several potential failure modes, as these will give an indication of potential crack and damage patterns. Four failure modes are possible, as described below.

i. Sliding-Shear Failure. The Mohr-Coulomb failure criteria can be used to assess the initial sliding-shear capacity of the infill:

$$V^{i}_{slide} = \left(\tau_0 + \sigma_y \tan\phi\right) L_{inf} t_{inf} = \mu N \qquad (8\text{-}4)$$

where τ_0 = cohesive capacity of the mortar beds, which, in the absence of data may be taken as

$$\tau_0 = \frac{f'_{me90}}{20} \qquad (8\text{-}5)$$

where ϕ = the angle of sliding friction of the masonry along a bed joint. Note that $\mu = \tan\phi$, where μ = coefficient of sliding friction along the bed joint. After the infill's cohesive bond strength is destroyed as a result of cyclic loading, the infill still has some ability to resist sliding through shear friction in the bed joints. As a result, the final Mohr-Coulomb failure criteria reduce to:

$$V^{i}_{slide} = \left(\sigma_y \tan\phi\right) L_{inf} t_{inf} = \mu N \qquad (8\text{-}6)$$

where N = vertical load in the panel. If deformations are small, then $V_{slide} \approx 0$ because σ_y may only result from the self-weight of the panel. However, if these interstory drifts become large, then the bounding columns impose a vertical load due to shortening of the height of the panel. The vertical shortening strain in the panel is given by

$$\varepsilon = \frac{\delta}{h} = \theta\frac{\Delta}{h} = \theta^2 \qquad (8\text{-}7)$$

where

- δ = downward movement of the upper beam as a result of the panel drift angle, θ
- h = interstory height (center-to-center of beams)
- Δ = interstory drift (displacement)
- θ = interstory drift angle

The axial load on the infill is

$$N = \varepsilon L_{inf} t_{inf} E_m \qquad (8\text{-}8)$$

where E_m = Young's modulus of the masonry, which in the absence of tests may be set at $550 f'_{me}$.

Substituting equations (8-7) and (8-8) into (8-6) gives

$$V^{i}_{slide} = \mu L_{inf} t_{inf} E_m \theta^2 \qquad (8\text{-}9)$$

ii. Compression Failure. For compression failure of the equivalent diagonal strut, a modified version of the method suggested by Stafford-Smith and Carter (1969) can be adopted. The shear force (horizontal component of the diagonal strut capacity) is calculated as

$$V_c = a t_{inf} f'_{m90} \cos\theta \qquad (8\text{-}10)$$

where

- a = equivalent strut width, defined above
- t_{inf} = infill thickness
- f'_{me90} = expected strength of masonry in the *horizontal* direction, which may be set at 50% of the expected stacked prism strength f'_{me}.

iii. Diagonal Tension Failure of Panel. Using the recommendation of Saneinejad and Hobbs (1995), the cracking shear in the infill is given by

$$V_{cr} = \frac{2\sqrt{2}\, t_{inf} \sigma_{cr}}{\left(\dfrac{L_{inf}}{h_{inf}} + \dfrac{h_{inf}}{L_{inf}}\right)} \qquad (8\text{-}11)$$

The cracking capacity of masonry, σ_{cr}, is somewhat dependent on the orientation of the principal stresses with respect to the bed joints.

In the absence of tests results, the cracking strength may be taken as

$$\delta_{cr} = \frac{f'_{me90}}{20} \qquad (8\text{-}12)$$

or

$$\sigma_{cr} \approx v_{me} \quad (8\text{-}13)$$

where v_{me} = cohesive strength of the masonry bed joint, which is given by

$$v_{me} = 20\sqrt{f'_{me}} \quad (8\text{-}14)$$

where f'_{me} = expected comprehensive strength of a masonry prism.

iv. General Shear Failure of Panel. Based on the recommendations of FEMA 273, as well as Paulay and Priestley (1992), the initial and final contributions of shear carried by the infill panel may be defined as:

$$v_{mi} = A_{vh} 2\sqrt{f'_{me}} \quad (8\text{-}15)$$

$$V_{mf} = 0.3V_{mi} \quad (8\text{-}16)$$

where

V_{mi} = available initial shear capacity that is consumed during the first half-cyclic (monotonic) loading

V_{mf} = final shear capacity as a result of cyclic-loading effects

A_{vh} = net horizontal shear area of the infill panel.

Note for a complete infill with no openings

$$A_{vh} = L_{inf} t_{inf} \quad (8\text{-}17)$$

The above values give upper and lower bounds to the cyclic-loading resistance of the infill.

v. The Effect of Infill Panel Reinforcement. If either a masonry or concrete infill panel is reinforced, then the reinforcement should improve the shear strength of the panel. The shear demand carried by the reinforcement is given by the well known ACI 318-95 (ACI, 1995) provisions.

$$v_s = \rho_w f_{ye} A_{vh} \quad (8\text{-}18)$$

where ρ_w = volumetric ratio of the reinforcement in the infill panel, f_{ye} = expected yield strength of the web reinforcement within the infill panel, and A_{vh} is defined above.

c. Deformation Capacity of Solid Infilled-Panel Components

There are no clear experimental results for the deformation capacities for each of the four behavior modes for infilled panel components, nor are there suitable analytical models available. Experiments show that diagonal cracking begins with the onset of nonlinear behavior at interstory drifts of 0.25% and is essentially complete (from corner to corner) in a panel by about 0.5%. Corner crushing begins at the same stage, but its extent depends on the amount of cyclic loading sustained. There is essentially no limit to the ability of an infill panel to deform in sliding shear—other behavior modes usually govern. Thus, limits imposed by the general shear behavior mode determine the displacement capacity of infill panels. Experimental evidence supports the following interstory drift limit states for different masonry infill panels:

Brick masonry	1.5%
Grouted concrete block masonry	2.0%
Ungrouted concrete block masonry	2.5%

8.3.2 Infilled-Panel Components with Openings

The strength of infill panels with openings is best assessed using rational models composed of subcomponents of the relevant materials. See Chapter 5 for concrete, Chapter 6 for reinforced masonry, and Chapter 7 for unreinforced masonry.

8.3.3 Out-of-Plane Behavior of Infilled-Panel Components

FEMA 273 as well as Angel and Abrams (1994) describe methods for assessing infill capacity to resist out-of-plane demands. Based on these recommendations, the following formulae can be used to assess the infill strength. In these expressions, w is the uniform pressure that causes out-of-plane failure of the wall.

$$w = \frac{2f'_{me}}{(h/t)} \lambda R_1 R_2 \quad (8\text{-}19)$$

where

f'_{me} = expected masonry strength
λ = slenderness parameter defined in Table 8-5
R_1 = out-of-plane reduction factors, set at $R_1 = 1$ for no damage (See Table 8-5 for moderate and severe damage)
R_2 = Stiffness-reduction factor for bending frame members, given by

$$R_2 = 0.35 + 71.4 \times (10)^{-9} EI \text{ not to exceed } 1 \quad (8\text{-}20)$$

where

EI = flexural rigidity of the weakest frame on the non-continuous side of the infill panel (units: k-in)

8.3.4 Steel-Frame Components

a. Flexure

The flexural strength of steel frames, based on conventional plastic concepts is given as

$$M_p = F_{ye} Z_x \quad (8\text{-}21)$$

where

Z_x = plastic-section modulus
F_{ye}= expected yield strength

In the absence of specific data, F_{ye} is set initially at 48 ksi (330 MPa) and 55 ksi (380 MPa) for Grades A36 and 50, respectively.

b. Shear

Steel-frame shear-strength capacity of beams and columns is based on the relationship

$$V_p = 0.6 A_w F_{ye} \quad (8\text{-}22)$$

where

F_{ye} = expected yield strength, with default values defined above
A_w = web area of the member

The 0.6 coefficient is based on the Von Mises yield criteria.

The critical section in a steel frame component with respect to shear is assumed to occur at an equivalent short beam or column formed between a frame joint and the compression strut associated with the infill panel. It may conservatively be assumed that the centroid of the diagonal strut force moves downward by an amount equal to

$$l_{ceff} = \frac{a}{\cos\theta_c} \quad (8\text{-}23)$$

where

a = effective width of a longitudinal compression struct. This is defined in Section Section 8.3.1a.
l_{ceff} = effective length of a "short" fixed-fixed column.

$$\tan\theta_c = \frac{h_{inf} - \left(\frac{a}{\cos\theta_c}\right)}{L_{inf}} \quad (8\text{-}24)$$

The shear demand is at a maximum when flexural plastic hinges form at each end of this so-called "short column", thus

$$V_{col} = \frac{2M_p^{col}}{l_{ceff}} \quad (8\text{-}25)$$

c. Joints

The strength and deformation capacity of riveted, bolted and welded connections along the panel zones are largely geometry-dependent. Due to wide variations in construction practice, the reader is referred to FEMA 273.

8.3.5 Concrete-Frame Components

a. Flexure

Flexural strength of reinforced concrete frames should be based on the nominal strength provisions of the ACI 318-95 code. However, expected strength values should be used for the material properties. Flexural deformation capacity depends on the amount of transverse reinforcement. If ductile detailing is used, then dependable plastic-hinge rotations of 0.035 radians can easily be attained. For nonductile detailing, experimental

Table 8-5 Out-of-plane infill strength parameters.

Height-to-thickness ratio $\frac{h}{t}$	Slenderness parameter λ	Strength-reduction factor R_I	
		Moderate Damage	*Severe Damage*
5	0.130	1.0	1.0
10	0.060	0.9	0.9
15	0.035	0.9	0.8
20	0.020	0.8	0.7
25	0.015	0.8	0.6
30	0.008	0.7	0.5
35	0.005	0.7	0.5
40	0.003	0.7	0.5

research by Aycardi et al. (1992, 1994) has shown that interstory drifts of 0.03 radians are possible. Because the drift demands on infilled-frame systems are generally not as great as for bare frames, it is likely that the rotational demands will be less than the rotational capacity.

b. Shear

The critical section for shear strength is similar to that for steel frames, as discussed in Section 8.3.4. If the "short column" member is shear-critical, then the corner-to-corner crack angle forms. This angle can be calculated from

$$\alpha_c = \tan^{-1}\frac{jd}{l_{ceff}} \tag{8-26}$$

where

jd = internal lever arm within the column member. In lieu of a precise analysis, this may be set at 80% of the overall member width.

Similarly, the beam should also be checked so that the effective length of the beam is given by

$$l_{ceff} = \frac{a}{\sin\theta_b} \tag{8-27}$$

$$V_b = \frac{\left(M_p^+ + M_p^-\right)}{l_{ceff}} \tag{8-28}$$

where

$$\tan\theta_b = \frac{L_{inf}}{L_{inf} - \dfrac{a}{\sin\theta_b}} \tag{8-29}$$

The corner-to-corner crack angle forming in a reinforced concrete beam is:

$$\alpha_b = \tan^{-1}\frac{d-d'}{l_{ceff}} \tag{8-30}$$

in which

l_{ceff} = "short-beam" length

$d - d'$ = distance between centroids of top and bottom reinforcement

M_p^+ = maximum positive moment generated by tensile yielding of the bottom reinforcement

M_p^- = maximum negative moment generated by tensile yielding of the top beam steel, including the effects of slab steel, if any.

$$M_p^+ = A_s\left(1.25 f_{ye}\right)(d-d') \tag{8-31}$$

$$M_p^- = A_s'\left(1.25 f_{ye}\right)(d-d') \tag{8-32}$$

where

$A_s =$ area of bottom steel. If this steel is not fully anchored and only extends a short distance into the joint, the value of A_s used in Equation 8-31 should be prorated by l_{em}/l_d where l_{em} = embedment length and l_d = development length, as given by ACI 318.

$A'_s =$ area of top steel, including slab steel

$1.25f_{ye} =$ expected overstrength of the tension reinforcement, the 1.25 factor allowing for strain-hardening effects, and f_y = probable/measured yield strength of the longitudinal beam reinforcement

Concrete-frame shear-strength capacity is initially based on the recommendations of ATC-40 and FEMA 273. This recommended design procedure generally provides a lower bound to the shear-strength capacity. The ultimate shear capacity is given by

$$V_u = V_s + V_c \tag{8-33}$$

where V_s is the shear carried by the steel

$$V_s = A_{sh} f_{yh} \frac{d}{s} \tag{8-34}$$

and V_c is the shear carried by concrete:

$$V_c = 3.5\lambda\left(k + \frac{N_u}{2000A_g}\right)\sqrt{f'_c}\,b_w d \tag{8-35}$$

where

- $k =$ 1.0 in regions of low ductility demand and 0 in regions of moderate or high ductility demand,
- $\lambda =$ 0.75 for lightweight aggregate concrete and 1.0 for normal-weight aggregate concrete
- $N_u =$ axial compression force in pounds (equals 0 for tension force)

The approach recommended by Priestley et al. (1996) is less conservative and may provide an estimate of shear capacity that is more compatible with observations in the field, particularly in the presence of large diagonal cracks. In this approach, the shear capacity is given by:

$$V_n = V_s + V_p + V_c \tag{8-36}$$

where V_s, V_p, and V_c are the shear demand carried by steel, compressive axial-strut force, and concrete mechanism, respectively. These are defined below.

V_s, the shear carried by the transverse reinforcement, is given by:

$$V_s = A_{sh} f_{yhe} \frac{jd}{s} \cot\theta \tag{8-37}$$

where

- $A_{sh} =$ area of steel in one transverse hoop set
- $f_{yhe} =$ expected strength of the transverse reinforcement
- $jd =$ internal lever arm, which in lieu of a more precise analysis may be set at 0.8D
- $D =$ member depth
- $s =$ center to center spacing of the hoop sets
- $\theta =$ corner-to-corner crack angle measured to the axis of the column

V_p, the shear demand carried by axial load (strut action) in a column, is given by

$$V_p = P\tan\theta \tag{8-38}$$

where

- $P =$ axial load in the frame member
- $\theta =$ as defined above.

V_c, the shear demand carried by the concrete is given by

$$V_c = k\sqrt{f'_{ce}}\,b_w d \tag{8-39}$$

in which

- $b_w =$ web width
- $d =$ effective member depth, and
- $k =$ coefficient depending on the displacement ductility of the member and may be defined as follows:

$$k = 3.5 \text{ for } \mu \le 2 \tag{8-40}$$

$$k = 1.2 \text{ for } \mu = 4 \tag{8-41}$$

$$k = 0.6 \text{ for } \mu \ge 8 \tag{8-42}$$

To find k for $2 < \mu < 4$ and $4 < \mu < 8$, use linear interpolation between the above-specified ductility limits. For equations containing f'_{ce}, use psi units. Note that upper- and lower-bound values of V_c should be computed, representing the initial and the final (residual) shear capacity values using Equations 8-40 and 8-42, respectively.

c. Joints

FEMA 273 presents guidelines for assessing beam-column joint strength using the formula given below:

$$V_n = \lambda\gamma\sqrt{f'_{ce}}A_j \tag{8-43}$$

where

$A_j =$ nominal gross-section area

$\lambda =$ 0.75 or 1.0 for light weight and normal weight aggregate concrete, respectively

$\gamma =$ strength coefficient ranging from 4 to 12 and 8 to 20 for joints without and with appreciable transverse reinforcement, respectively.

Specific values of γ may be found in Table 6-8 of FEMA 273.

As an alternative to the FEMA 273 approach, the following procedure used in bridge-joint evaluation (Priestley, 1996) may be helpful for correlating behavior modes and observed damage patterns.

The nominal principal stresses on a joint are used to assess whether the joint will crack. A stress analysis that employs Mohr's circle is used to determine the major principal tension stress

$$\sigma_t = \frac{\sigma_x + \sigma_y}{2} + \sqrt{v_j^2 + \left(\frac{\sigma_x + \sigma_y}{2}\right)^2} \tag{8-44}$$

where σ_x, σ_y and v_j are the average bounding actions on the joint, as defined below.

σ_y is the average normal stress on the column given by:

$$\sigma_y = \frac{P_{col}}{b_c h_c} \tag{8-45}$$

where

$P_{col} =$ column axial load (tension positive)

$b_c =$ column depth

$h_c =$ column width

σ_x is the average normal stress on the beam, given by:

$$\sigma_x = \frac{P_b}{d_b b_b} \tag{8-46}$$

in which

$d_b =$ overall beam depth

$b_b =$ beam width

$P_b =$ axial force in the beam (if any)

v_j, the average joint shear stress is given by:

$$v_j = \frac{V_{jh}}{b_j h_c} \tag{8-47}$$

in which

$V_{jh} =$ horizontal joint shear force assuming column and beam

$b_j =$ smaller of b_c or b_b

$$\text{If: } \sigma_t < 3.5\sqrt{f'_{ce}} \tag{8-48}$$

then the joint may be assumed to remain elastic and uncracked.

$$\text{If: } \sigma_t > 5\sqrt{f'_{ce}} \tag{8-49}$$

$$\text{or } \sigma_t > 7\sqrt{f'_{ce}} \tag{8-50}$$

then full diagonal cracking may be expected for exterior joints or corner joints, respectively, under biaxial response.

If σ_t is between the above limits, then some partial cracking may be expected. Moreover, if hinging occurs in the beam adjacent to the connection, then yield penetration of the longitudinal bars into the joint occurs. With sequential cycling, this eventually leads to joint failure.

Similarly, the principal compression stress, σ_c, should be checked such that

$$\sigma_c = \frac{\sigma_x + \sigma_y}{2} - \sqrt{v_j^2 + \left(\frac{\sigma_x + \sigma_y}{2}\right)^2} \qquad (8\text{-}51)$$

If, for one-way frames

$$|\sigma_c| > 0.5 f'_{ce} \qquad (8\text{-}52)$$

or, if for two-way joints where biaxial loading may occur

$$|\sigma_c| > 0.45 f'_{ce} \qquad (8\text{-}53)$$

then joint failure may be expected due to compression crushing of the diagonal struts within the joints.

Degradation of joint strength after crack formation may be expected. For assessing the degraded strength, the following rules may be used:

$$\gamma_j < 0.005 \quad \text{no change in } \sigma_t \qquad (8\text{-}54)$$

$$\gamma_j < 0.02 \quad \sigma_t = 1.2\sqrt{f'_{ce}} \qquad (8\text{-}55)$$

$$\gamma_j < 0.04 \quad \sigma_t = 0 \qquad (8\text{-}56)$$

where

$\gamma_j =$ joint rotation angle (in radians)

For intermediate values of γ_j linear interpolation may be used to determine σ_t. With this value for σ_t, the joint shear strength may be determined from Equations 8-44 and 8-47 as follows:

$$V_{jh} = b_j h_c \sqrt{\left(\sigma_t - \frac{\sigma_x + \sigma_y}{2}\right) - \left(\frac{\sigma_x + \sigma_y}{2}\right)^2} \qquad (8\text{-}57)$$

d. Bond Slip of Lap-Splice Connections

Lap-splice connections often occur at the base of a column, particularly in older nonductile concrete frames. Provided that the lap length is sufficient to develop yield (i.e., $20d_b$), the nominal ultimate strength capacity can be attained. However, postelastic deformations quickly degrade the bond-strength capacity, and within one inelastic cycle of loading, the lap splice may be assumed to have failed. This failure is evident if longitudinal (tensile) splitting cracks are noticed at the base of the column.

When the lap splice fails in bond, it does not generally lead to a catastrophic failure, as the column is still able to transfer moment due to the presence of the eccentric compression stress block that arises as a result of the axial load in the column. Thus, the following initial and final failure model may be assumed:

$$M_i = M_n \ \text{for} \ \theta_p = 0 \qquad (8\text{-}58)$$

and

$$M_f = P\left(\frac{d - d'}{2}\right) \ \text{for} \ \theta_p > 0.025 \qquad (8\text{-}59)$$

where

$\theta_p =$ plastic rotation of the connection in radians

$P =$ axial load that takes into account the variation in force due to the truss action of infilled frames. Note that axial load will increase for a compression side column, whereas for a tension side column the axial load will decrease to a point that tension could be induced. For the latter case assume $P = 0$.

$(d - d') =$ distance between the outer layers of reinforcement in the column.

For intermediate values of plastic hinge rotation ($\theta_p > 0.025$) the following interpolation may be used

$$M_I = M_n - \frac{\theta_p}{0.025}\left(M_n - M_f\right) \qquad (8\text{-}60)$$

At large rotations the concrete crushes and may be severely damaged.

8.4 Infilled Frame Component Guides

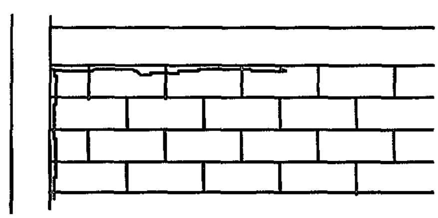
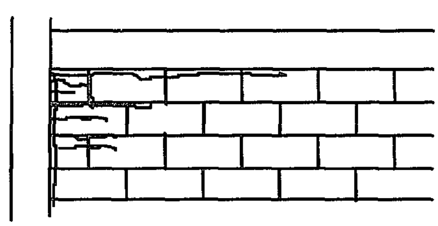
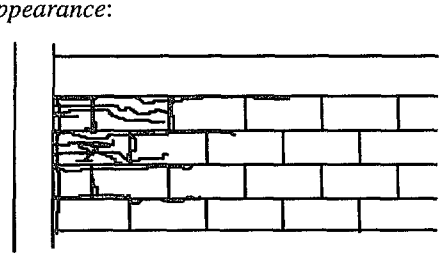
The following Component Damage Classification Guides contain details of the behavior modes for infilled-frame components. Included are the distinguishing characteristics of the specific behavior mode, the description of damage at various levels of severity, and performance restoration measures. Information may not be included in the Component Damage Classification Guides for certain damage severity levels; in these instances, for the behavior mode under consideration, it is not possible to make refined distinctions with regard to severity of damage. See also Section 3.5 for general discussion of the use of the Component Guides and Section 4.4.3 for information on the modeling and acceptability criteria for components.

INPS1	COMPONENT DAMAGE CLASSIFICATION GUIDE	System:	**Infilled Frame**
		Component Type:	**Infill Panel**
		Behavior Mode:	**Corner crushing**
		Applicable Materials:	**Concrete Frame-Block Infill**

How to Distinguish Damage Type:

By Observation:
This type of damage occurs because ungrouted concrete block infills are inherently weak compared to adjacent concrete columns. Lateral movements create high corner strains leading to early failure of the corner concrete blocks. Some diagonal cracking and/or concrete bed joint sliding is also evident.

By Analysis (See Section 8.3):
The plastic limit methods of Liauw and Kwan (1983) or the simplified truss method of Stafford-Smith (1966) can be used to analyze the hierarchy of strength mechanisms.

Severity	Description of Damage	Performance Restoration Measures
Insignificant $\lambda_K = 0.9$ $\lambda_Q = 0.9$ $\lambda_D = 1.0$	*Criteria:* Separation of mortar around perimeter of panel and some crushing or mortar near corners of infill panel *Typical Appearance:*. 	• Repoint spalled mortar. • Inject cracks.
Moderate $\lambda_K = 0.6$ $\lambda_Q = 0.8$ $\lambda_D = 0.8$	*Criteria*: Crushing of mortar, cracking of blocks including lateral movement of face shells. *Typical Appearance*: 	• Remove and replace damaged units. • Inject cracks around perimeter of infill. • Apply composite overlay at damaged corners.
Heavy $\lambda_K = 0.5$ $\lambda_Q = 0.7$ $\lambda_D = 0.7$	*Criteria*: Loss of corner blocks through complete spalling of face shells. Diagonal (stairstep) cracking and/or bed joint sliding may also be evident. *Typical Appearance*: 	• Remove and replace infill or apply composite overlay on infill.

INPS2

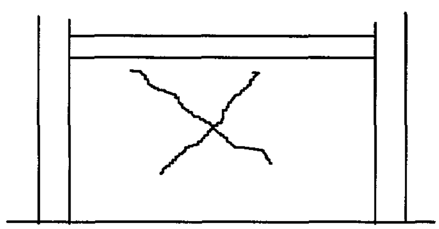
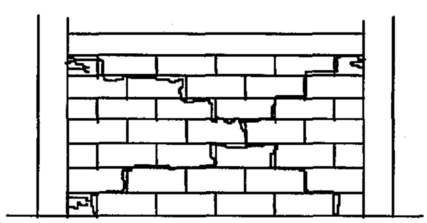
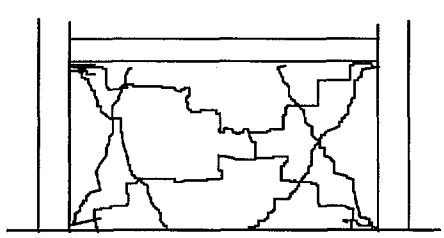
COMPONENT DAMAGE CLASSIFICATION GUIDE

System:	**Infilled Frame**
Component Type:	**Infill Panel**
Behavior Mode:	**Diagonal Tension**
Applicable Materials:	**Masonry Units**

How to distinguish damage type:

By Observation:
In this type of damage, cracking occurs across the diagonals of the infill panel. If drifts are large, secondary cracking may also be expected at an angle of about 45 degrees to 65 degrees to the horizontal. For large drifts the corner strains are intense and crushing may also be observed.

By Analysis (See Section 8.3):
It is possible to determine the diagonal cracking strength by rational mechanics, or by simplified strut methods such as that proposed by Stafford-Smith et al. (1969).

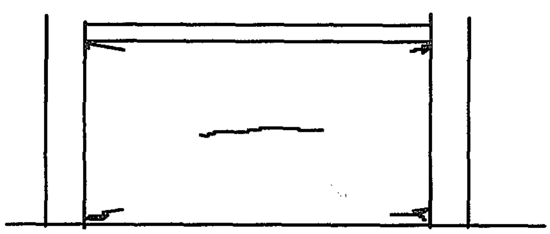
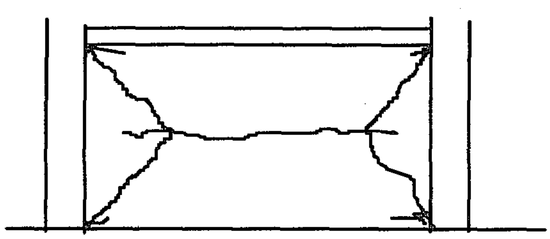
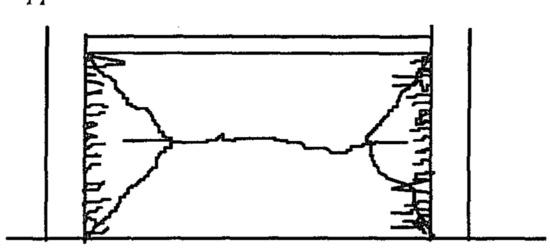
Severity	Description of Damage	Performance Restoration Measures
Insignificant $\lambda_K = 0.7$ $\lambda_Q = 0.9$ $\lambda_D = 1.0$	*Criteria:* Initial hairline cracking occur on diagonals in masonry. This is mostly associated with breaking of the bond between mortar and bricks. Cracking mostly concentrated within center region of panel *Typical appearance:*. 	Measures not necessary for structural performance restoration. (Certain measures may be necessary for the restoration of nonstructural characteristics).
Moderate $\lambda_K = 0.4$ $\lambda_Q = 0.8$ $\lambda_D = 0.9$	*Criteria:* Hairline cracks fully extend along diagonals following the mortar courses in a stairstep fashion, but sometimes propagate through bricks. Some crushing and/or "walking-out" of the mortar may be observed. Cracks mostly closed due to confinement provided by frames. *Typical appearance* 	Repoint spalled mortar. Remove and replace damaged masonry units.
Heavy $\lambda_K = 0.2$ $\lambda_Q = 0.5$ $\lambda_D = 0.8$	*Criteria:* Cracks widen to about 1/8", and are usually associated with corner crushing. Much loss of mortar is evident. More than one diagonal crack is generally evident. Crushing/cracking of the bricks is also evident. Portions of the entire infill may "walk" out-of-plane under cyclic loading. *Typical appearance:* 	Remove and replace damaged infill, or patch spalls. Apply shotcrete, ferrocement or composite overlay.

INPS3	COMPONENT DAMAGE CLASSIFICATION GUIDE	System:	**Infilled Frame**
		Component Type:	**Infill Panel**
		Behavior Mode:	**Bed joint sliding.**
		Applicable Materials:	**Steel-Frame Brick Infill**

How to Distinguish Damage Type:

By Observation:
In this type of damage, crushing may initially occur at the corner of the infill. As the displacements become large to accommodate racking movements in the panel, movements occur along bed joints in the form of diagonal (stairstep) cracking or horizontal bed joint sliding. Such movements are a secondary outcome to corner crushing.

By Analysis (See Section 8.3):
The effective strut method of analysis suggested by Stafford-Smith (1966) and/or plastic limit methods suggested by Liauw and Kwan (1983) should be used to check the hierarchy of failure mechanisms.

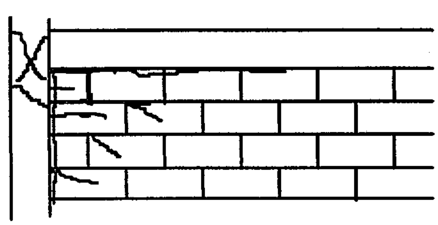
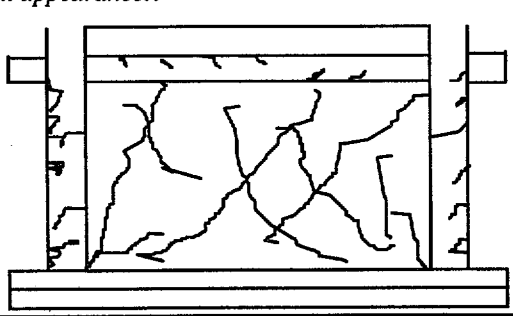
Severity	Description of Damage	Performance Restoration Measures
Insignificant $\lambda_K = 0.8$ $\lambda_Q = 0.9$ $\lambda_D = 0.9$	*Criteria:* Crushing of mortar around perimeter of frame. This is particularly noticeable adjacent to the columns near the corners of the infill panels. *Typical appearance:* 	Repoint spalled mortar. Inject cracks.
Moderate $\lambda_K = 0.5$ $\lambda_Q = 0.8$ $\lambda_D = 0.8$	*Criteria:* Crushing of mortar and cracking of bricks extend over larger zones adjacent to beam and column *Typical appearance:*. 	Remove damaged bricks and replace.
Heavy $\lambda_K = 0.4$ $\lambda_Q = 0.7$ $\lambda_D = 0.7$	*Criteria:* Significant crushing of mortar and bricks extends around most of the perimeter frame, particularly along the height of the column. *Typical appearance:* 	Remove and replace infill, or patch spalls. Apply shotcrete, ferrocement or composite overlay on infill.

INPS4	COMPONENT DAMAGE CLASSIFICATION GUIDE	System:	**Infilled Frame**
		Component Type:	**Infill Panel**
		Behavior Mode:	**Corner crushing <u>and</u> diagonal cracking**
		Applicable Materials:	**Concrete Frame-Block Infill**

How to Distinguish Damage Type:

By Observation:
Damage is equally distributed between both the frame and the infill. Crushing of the blocks and severe flexural cracking of infill occurs. Distributed diagonal cracks also occur.

By Analysis (See Section 8.3):
Limit-strength analysis methods are necessary to determine strength distribution well into the inelastic range.

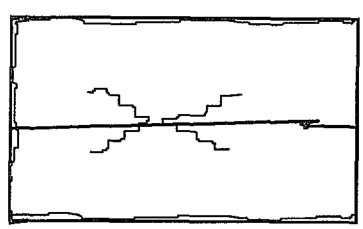
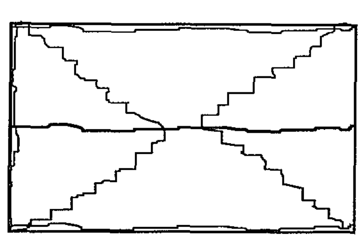
Severity	Description of Damage	Performance Restoration Measures
Insignificant $\lambda_K = 0.9$ $\lambda_Q = 1.0$ $\lambda_D = 1.0$	*Criteria:* Separation of mortar around frame occurs first in beam-to-infill interface. Some hairline cracks may be evident along mortar courses.	Repoint spalled mortar. Inject cracks.
Moderate $\lambda_K = 0.6$ $\lambda_Q = 0.8$ $\lambda_D = 0.8$	*Criteria:* For a ductile (strong column-weak beam) frame design, yielding of longitudinal reinforcement occurs first in beam, with minor cracking in columns. Compression splitting occurs in corner blocks. Some hairline X cracks may be expected in beam-column joint *Typical appearance:.* 	Remove and replace damaged masonry units. Inject cracks.
Heavy $\lambda_K = 0.5$ $\lambda_Q = 0.6$ $\lambda_D = 0.6$	*Criteria:* Extensive cracking in beam and column hinge zones, leading to spalling of cover concrete in frame. Diagonal cracking passes through blocks. Faceshells spall off in corners, and also across a critical shear plane at mid-height of infill *Typical appearance:.* 	Remove and replace infill. Remove and patch spalled and loose concrete in frame. Inject cracks.

INPS5	COMPONENT DAMAGE CLASSIFICATION GUIDE	System: **Infilled Frame**
		Component Type: **Infill Panel**
		Behavior Mode: **Out-of-Plane**
		Applicable Materials: **Masonry Infill**

How to Distinguish Damage Type:

By Inspection:
This type of damage occurs with strong out-of-plane shaking. When coupled with in-plane shaking, the panel could potentially fall out. This behavior makes it difficult to distinguish which of the two types of shaking caused the damage.

By Analysis (See Section 8.3):
Arching action analysis is necessary.

Severity	Description of Damage	Performance Restoration Measures
Insignificant $\lambda_K = 0.9$ $\lambda_Q = 1.0$ $\lambda_D = 1.0$	*Criteria:* Flexural cracking in the mortar beds around the perimeter, with hairline cracking in mortar bed at mid-height of panel.	Repoint spalled mortar.
Moderate $\lambda_K = 0.9$ $\lambda_Q = 0.8$ $\lambda_D = 1.0$	*Criteria:* Crushing and loss of mortar along top, mid-height, bottom and side mortar beds. Possibly some in-plane damage, as evidenced by hair-line X-cracks in the central panel area. *Typical appearance:* 	Apply shotcrete, ferrocement, or composite overlay to the infill.
Heavy $\lambda_K = 0.5$ $\lambda_Q = 0.6$ $\lambda_D = 0.9$	*Criteria:* Severe corner-to-corner cracking with some out-of-plane dislodgment of masonry. Top, bottom and mid-height mortar bed is completely crushed and/or missing. There is some out-of-plane dislodgment of masonry. Concurrent in-plane damage should also be expected, as evidenced by extensive X-cracking. *Typical appearance:* 	Remove and replace infill.

INF1C1	COMPONENT DAMAGE CLASSIFICATION GUIDE		
		System:	**Infilled Frame**
		Component Type:	**Concrete Column**
		Behavior Mode:	**Column Snap through Shear Failure**
		Applicable Materials:	**Concrete Frame Masonry Infill**

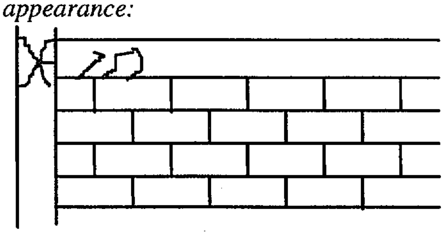
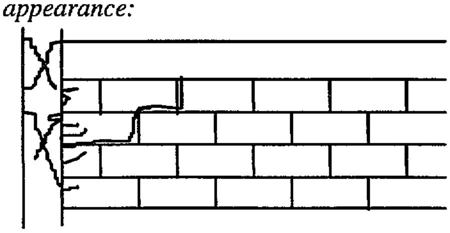
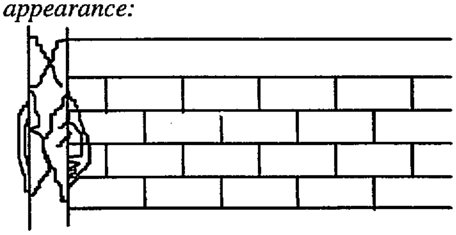
How to Distinguish Damage Type:

By Observation:

If infills are stiff and/or strong, then the frame is the weaker component. Cracking is not across a corner-to-corner diagonal, but on a flatter angle. Column cracking over a length equal to two member widths is severe and a sign of low frame shear capacity.

By Analysis (See Section 8.3):

Column shear capacity should be checked. Shear failure is generally associated with inadequate transverse (shear/confinement) reinforcement.

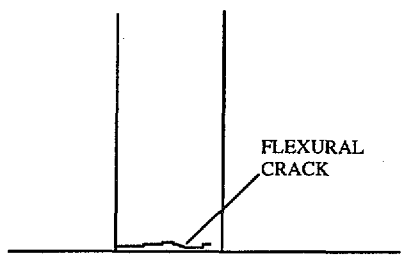
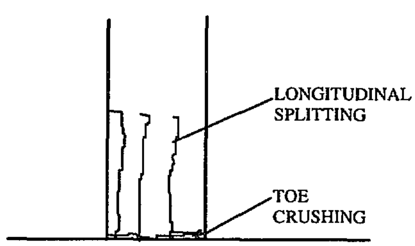
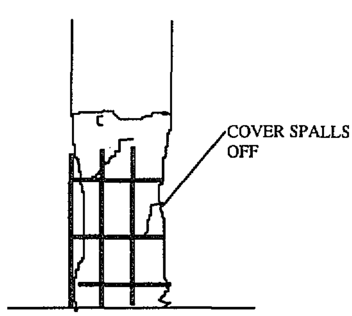
Severity	Description of Damage	Performance Restoration Measures
Insignificant $\lambda_K = 0.9$ $\lambda_Q = 0.9$ $\lambda_D = 1.0$	*Criteria:* Several flexural cracks form in columns near top corner of infill. *Typical appearance:* 	Remove and patch spalled and loose concrete. Inject cracks.
Moderate $\lambda_K = 0.7$ $\lambda_Q = 0.7$ $\lambda_D = 0.4$	*Criteria:* Flexure cracks change into shear X-cracks over a short length near column end. (Generally over about two column widths). Column cover in this vicinity will be loose. Some associated crushing may appear in the infill. *Typical appearance:* 	Remove and patch spalled and loose concrete. Apply composite overlay to damaged region of column.
Heavy $\lambda_K = 0.4$ $\lambda_Q = 0.2$ $\lambda_D = 0.4$	Criteria: Cracking in column may be so severe that transverse hoops have fractured about one member width away from column end (at middle of X-cracks). Cover concrete in this vicinity will be mostly spalled away. *Typical appearance:* 	Remove spalled and loose concrete. Remove and replace buckled or fractured reinforcing. Provide additional ties over length of replaced bars. Patch concrete. Inject cracks. Apply composite overlay to damaged region of column.

INF1C2	COMPONENT DAMAGE CLASSIFICATION GUIDE	System:	**Infilled Frame**
		Component Type:	**Concrete Column**
		Behavior Mode:	**Lap Splice Failure**
		Applicable Materials:	**Reinforced Concrete**

How to Distinguish Damage Type:

<u>By Inspection:</u>
Lack of sufficient lap length in hinge zone leads to eventual slippage. The cover spalls off due to high compression stresses, exposing the core concrete and damaged lap splice zone.

<u>By Analysis (See Section 8.3):</u>
Refer to FEMA 307.

Severity	Description of Damage	Performance Restoration Measures
Insignificant $\lambda_K = 0.9$ $\lambda_Q = 1.0$ $\lambda_D = 1.0$	*Criteria:* Flexural cracks at floor level. Slight hairline vertical cracks. *Typical appearance* 	Inject cracks in frame.
Moderate $\lambda_K = 0.8$ $\lambda_Q = 0.5$ $\lambda_D = 1.0$	*Criteria:* Tensile flexural cracks at floor slab level with some evidence of toe crushing over the bottom 1/2". Longitudinal splitting cracks loosen the cover concrete. *Typical appearance:* 	Inject cracks in frame.
Heavy $\lambda_K = 0.5$ $\lambda_Q = 0.5$ $\lambda_D = 1.0$	*Criteria:* Significant spalling of the cover concrete over the length of the lap splice, exposing the core and reinforcing steel *Typical appearance:.* 	Remove spalled and loose concrete. Provide additional ties over the length of the exposed bars. Patch concrete. Apply composite overlay to damaged region of column.

INF3C

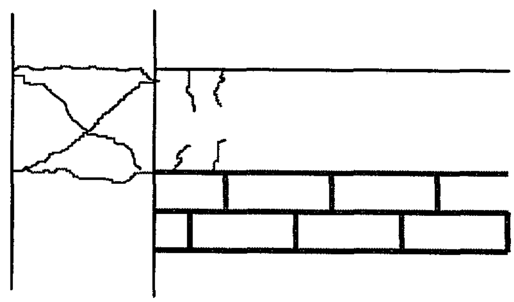
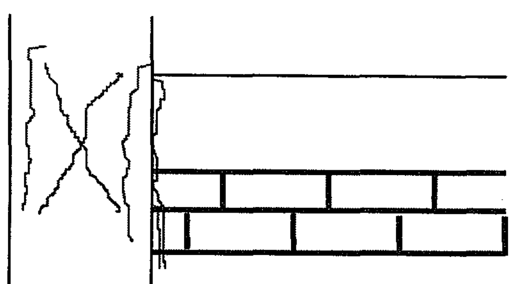
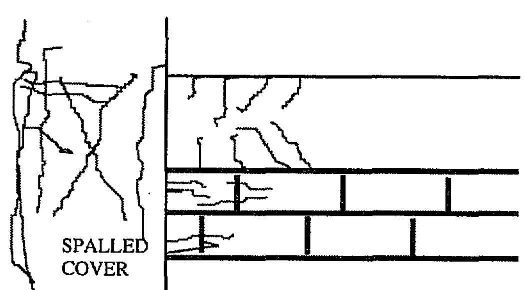
COMPONENT DAMAGE CLASSIFICATION GUIDE

System:	**Infilled Frame**
Component Type:	**Concrete Frame**
Behavior Mode:	**Connection Damage**
Applicable Materials:	**Reinforced Concrete**

How to Distinguish Damage Type:

By Inspection:
Distress is caused by overstrength of members framing into the connection, leading to very high principal tension stresses.

By Analysis (See Section 8.3):
Refer to FEMA 307.

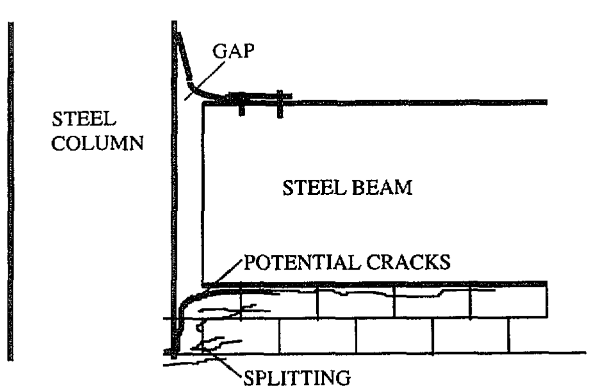
Severity	Description of Damage	Performance Restoration Measures
Insignificant $\lambda_K = 0.9$ $\lambda_Q = 1.0$ $\lambda_D = 1.0$	*Criteria:* Slight X hairline cracks in joint *Typical appearance:.* 	Inject cracks.
Moderate $\lambda_K = 0.8$ $\lambda_Q = 0.5$ $\lambda_D = 0.9$	*Criteria:* X-cracks in joint become more extensive and widen to about 1/8". *Typical appearance:* 	Inject cracks.
Heavy $\lambda_K = 0.5$ $\lambda_Q = 0.5$ $\lambda_D = 0.5$	*Criteria:* Extensive X-cracks in joint widen to about 1/4". Exterior joints show cover concrete spalling off from back of joint. Some side cover may also spall off. *Typical appearance:* 	Remove spalled and loose concrete. Remove and replace buckled or fractured reinforcing. Provide additional ties over the length of the replaced bars. Patch concrete. Inject cracks.

INF3S	COMPONENT DAMAGE CLASSIFICATION GUIDE	System:	**Infilled Frame**
		Component Type:	**Framed Connection**
		Behavior Type:	**Simple Connection Damage**
		Applicable Materials:	**Steel Frame-Masonry Infill**

How to Distinguish Damage Type:

By Observation:
Damage to simple (semi-rigid) steel connections occur due to the high shears that must be transferred in the inelastic range.

By Analysis (See Section 8.3):
Plastic limit analysis of connections (see Mander et al. (1994)) is necessary. Fatigue failure is unlikely, but should be checked.

Severity	**Description of Damage**	**Performance Restoration Measures**
Insignificant $\lambda_K = 0.9$ $\lambda_Q = 1.0$ $\lambda_D = 1.0$	*Criteria:* As the frame racks, the connection yields and paint may flake off.	Unnecessary for restoration of structural performance. (Certain measures may be necessary for restoration of nonstructural characteristics).
Moderate $\lambda_K = 0.9$ $\lambda_Q = 1.0$ $\lambda_D = 1.0$	*Criteria:* As drifts increase, both prying and slip may be evident. Angles pull away from column face, leaving infill frame bay with a larger overall opening. Gaps may be apparent around the perimeter of the infill.	Repaint and retorque bolts if loosened.
Heavy $\lambda_K = 0.5$ $\lambda_Q = 1.0$ $\lambda_D = 0.9$	*Criteria:* Angles may show fatigue cracks or failure. If this is the case, the infill will also be showing signs of significant distress.  	Remove angles and replace both bolts and angles. Remove infill and replace.

Glossary

Bearing wall A concrete or masonry wall that supports a portion of the building weight, in addition to its own weight, without a surrounding frame.

Behavior mode The predominant type of damage observed for a particular component. This is dependent on the relative magnitudes of the ratios of applied loads to component strength for axial, flexural and shearing actions.

Collapse Prevention A performance level whereby a building is extensively damaged, has little residual stiffness and strength, but remains standing; any other damage is acceptable.

Component A structural member such as a beam, column, or wall, that is an individual part of a structural element.

Coupled wall A wall element in which vertical pier components are joined at one or more levels by horizontal spandrel components.

Damage Physical evidence of inelastic deformation of a structural component caused by a damaging earthquake.

Damaging ground motion The ground motion that shook the building under consideration and caused resulting damage. This ground motion may or may not have been recorded at the site of the building. In some cases, it may be an estimate of the actual ground motion that occurred. It might consist of estimated time-history records or corresponding response spectra.

Direct method The determination of performance restoration measures from the observed damage without relative performance analysis.

Element An assembly of structural components (e.g., coupled shear walls, frames).

Global displacement capacity The maximum global displacement tolerable for a specific performance level. This global displacement limit is normally controlled by the acceptability of distortion of individual components or a group of components within the structure.

Global performance displacement demand The overall displacement of a representative point on a building subject to a performance ground motion. The representative point is normally at the roof level or at the effective center of mass for a given mode of vibration.

Global structure The assembly representing all of the structural elements of a building.

Immediate Occupancy A performance level whereby a building sustains minimal or no damage to its structural elements and only minor damage to its nonstructural components.

Inelastic lateral mechanism The plastic mechanism formed in an element, or assembly of elements, under the combined action of vertical and lateral loads. This is a unique mechanism for a specified pattern of lateral loads.

Infilled frame A concrete or steel frame with concrete or masonry panels installed between the beams and columns.

Life Safety A performance level whereby a building may experience extensive damage to structural and nonstructural components, but remains stable and has significant reserve capacity.

Nonlinear static procedure A structural analysis technique in which the structure is modeled as an assembly of components capable of nonlinear force-deformation behavior, then subjected to a monotonically increasing lateral load in a specific pattern to generate a global force-displacement capacity curve. The displacement demand is determined with a spectral representation of ground motion using one of several alternative methods.

Performance ground motion Hypothetical ground motion consistent with the specified seismic hazard level associated with a specific performance objective. This is characterized by time history record(s) or corresponding response spectra.

Performance objective A goal consisting of a specific performance level for a building subject to a specific seismic hazard.

Performance level A hypothetical damage state for a building used to establish design seismic performance objectives. The most common performance levels, in order of decreasing amounts of damage, are Collapse Prevention, Life Safety, and Immediate Occupancy.

Performance restoration measures Actions that might be implemented for a damaged building that result in future performance equivalent to that of the building in its pre-event state for a specific performance objective. These hypothetical repairs would result in a restored performance index equal to the performance index of the pre-event building.

Pier A vertical wall component.

Pre-existing condition Physical evidence of inelastic deformation or deterioration of a structural component that existed before the damaging earthquake

Relative performance analysis An analysis of a building in its damaged and pre-event condition to determine the effects of the damage on the capability of the building to meet specific seismic performance objectives.

Repair An action taken to address a damaged component of a building.

Severity of damage The relative intensity of damage to a particular component classified as insignificant, slight, moderate, heavy, or extreme.

Shear wall A concrete or masonry panel, connected to the adjacent floor system, that resists in-plane lateral loads.

Spandrel A wall component that spans horizontally.

Structural repairs Repairs that address damage to components to restore structural properties.

List of General Symbols

d_c	Global displacement capacity for pre-event structure for specified performance level.
d_c'	Global displacement capacity for damaged structure for specified performance level.
d_c^*	Global displacement capacity for repaired structure for specified performance level.
d_d	Global displacement demand for pre-event structure for specified seismic hazard.
d_d'	Global displacement demand for damaged structure for specified seismic hazard.
d_d^*	Global displacement demand for repaired structure for specified seismic hazard.
d_e	Maximum global displacement caused by the damaging ground motion.
λ_D	Modification factor applied to component deformation acceptability limits to account for earthquake damage.
λ_K	Modification factor for idealized component force-deformation curve to account for change in effective initial stiffness resulting from earthquake damage.
λ_Q	Modification factor for idealized component force-deformation curve to account for change in expected strength resulting from earthquake damage.
RD	Absolute value of the residual deformation in a structural component resulting from earthquake damage.
Q_{CE}	Expected strength of a component or element at the deformation level under consideration in a deformation-controlled element.

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Applied Technology Council Projects And Report Information

One of the primary purposes of Applied Technology Council is to develop resource documents that translate and summarize useful information to practicing engineers. This includes the development of guidelines and manuals, as well as the development of research recommendations for specific areas determined by the profession. ATC is not a code development organization, although several of the ATC project reports serve as resource documents for the development of codes, standards and specifications.

Applied Technology Council conducts projects that meet the following criteria:

1. The primary audience or benefactor is the design practitioner in structural engineering.
2. A cross section or consensus of engineering opinion is required to be obtained and presented by a neutral source.
3. The project fosters the advancement of structural engineering practice.

A brief description of several major completed projects and reports is given in the following section. Funding for projects is obtained from government agencies and tax-deductible contributions from the private sector.

ATC-1: This project resulted in five papers that were published as part of *Building Practices for Disaster Mitigation, Building Science Series 46*, proceedings of a workshop sponsored by the National Science Foundation (NSF) and the National Bureau of Standards (NBS). Available through the National Technical Information Service (NTIS), 5285 Port Royal Road, Springfield, VA 22151, as NTIS report No. COM-73-50188.

ATC-2: The report, *An Evaluation of a Response Spectrum Approach to Seismic Design of Buildings*, was funded by NSF and NBS and was conducted as part of the Cooperative Federal Program in Building Practices for Disaster Mitigation. Available through the ATC office. (Published 1974, 270 Pages)

ABSTRACT: This study evaluated the applicability and cost of the response spectrum approach to seismic analysis and design that was proposed by various segments of the engineering profession. Specific building designs, design procedures and parameter values were evaluated for future application. Eleven existing buildings of varying dimensions were redesigned according to the procedures.

ATC-3: The report, *Tentative Provisions for the Development of Seismic Regulations for Buildings* (ATC-3-06), was funded by NSF and NBS. The second printing of this report, which includes proposed amendments, is available through the ATC office. (Published 1978, amended 1982, 505 pages plus proposed amendments)

ABSTRACT: The tentative provisions in this document represent the results of a concerted effort by a multi-disciplinary team of 85 nationally recognized experts in earthquake engineering. The provisions serve as the basis for the seismic provisions of the 1988 *Uniform Building Code* and the 1988 and subsequent issues of the *NEHRP Recommended Provisions for the Development of Seismic Regulation for New Buildings.* The second printing of this document contains proposed amendments prepared by a joint committee of the Building Seismic Safety Council (BSSC) and the NBS.

ATC-3-2: The project, Comparative Test Designs of Buildings Using ATC-3-06 Tentative Provisions, was funded by NSF. The project consisted of a study to develop and plan a program for making comparative test designs of the ATC-3-06 Tentative Provisions. The project report was written to be used by the Building Seismic Safety Council in its refinement of the ATC-3-06 Tentative Provisions.

ATC-3-4: The report, *Redesign of Three Multistory Buildings: A Comparison Using ATC-3-06 and 1982 Uniform Building Code Design Provisions*, was published under a grant from NSF. Available through the ATC office. (Published 1984, 112 pages)

ABSTRACT: This report evaluates the cost and technical impact of using the 1978 ATC-3-06 report, *Tentative Provisions for the Development of Seismic Regulations for Buildings*, as amended by a joint committee of the Building Seismic Safety Council and the National Bureau of Standards in 1982. The evaluations are based on studies of three existing California buildings redesigned in accordance with the ATC-3-06 Tentative Provisions and the 1982 *Uniform Building Code*. Included in the report are recommendations to code implementing bodies.

ATC-3-5: This project, Assistance for First Phase of ATC-3-06 Trial Design Program Being Conducted by the Building Seismic Safety Council, was funded by the Building Seismic Safety Council to provide the services of the ATC Senior Consultant and other ATC personnel to assist the BSSC in the conduct of the first phase of its Trial Design Program. The first phase provided for trial designs conducted for buildings in Los Angeles, Seattle, Phoenix, and Memphis.

ATC-3-6: This project, Assistance for Second Phase of ATC-3-06 Trial Design Program Being Conducted by the Building Seismic Safety Council, was funded by the Building Seismic Safety Council to provide the services of the ATC Senior Consultant and other ATC personnel to assist the BSSC in the conduct of the second phase of its Trial Design Program. The second phase provided for trial designs conducted for buildings in New York, Chicago, St. Louis, Charleston, and Fort Worth.

ATC-4: The report, *A Methodology for Seismic Design and Construction of Single-Family Dwellings*, was published under a contract with the Department of Housing and Urban Development (HUD). Available through the ATC office. (Published 1976, 576 pages)

ABSTRACT: This report presents the results of an in-depth effort to develop design and construction details for single-family residences that minimize the potential economic loss and life-loss risk associated with earthquakes. The report: (1) discusses the ways structures behave when subjected to seismic forces, (2) sets forth suggested design criteria for conventional layouts of dwellings constructed with conventional materials, (3) presents construction details that do not require the designer to perform analytical calculations, (4) suggests procedures for efficient plan-checking, and (5) presents recommendations including details and schedules for use in the field by construction personnel and building inspectors.

ATC-4-1: The report, *The Home Builders Guide for Earthquake Design*, was published under a contract with HUD. Available through the ATC office. (Published 1980, 57 pages)

ABSTRACT: This report is an abridged version of the ATC-4 report. The concise, easily understood text of the Guide is supplemented with illustrations and 46 construction details. The details are provided to ensure that houses contain structural features that are properly positioned, dimensioned and constructed to resist earthquake forces. A brief description is included on how earthquake forces impact on houses and some precautionary constraints are given with respect to site selection and architectural designs.

ATC-5: The report, *Guidelines for Seismic Design and Construction of Single-Story Masonry Dwellings in Seismic Zone 2*, was developed under a contract with HUD. Available through the ATC office. (Published 1986, 38 pages)

ABSTRACT: The report offers a concise methodology for the earthquake design and construction of single-story masonry dwellings in Seismic Zone 2 of the United States, as defined by the 1973 *Uniform Building Code*. The Guidelines are based in part on shaking table tests of masonry construction conducted at the University of California at Berkeley Earthquake Engineering Research Center. The report is written in simple language and includes basic house plans, wall evaluations, detail drawings, and material specifications.

ATC-6: The report, *Seismic Design Guidelines for Highway Bridges*, was published under a contract with the Federal Highway Administration (FHWA). Available through the ATC office. (Published 1981, 210 pages)

ABSTRACT: The Guidelines are the recommendations of a team of sixteen nationally recognized experts that included consulting engineers, academics, state and federal agency representatives from throughout the United States. The Guidelines embody several new concepts that were significant departures from then existing design provisions. Included in the Guidelines are an extensive commentary, an example demonstrating the use of the

Guidelines, and summary reports on 21 bridges redesigned in accordance with the Guidelines. The guidelines have been adopted by the American Association of Highway and Transportation Officials as a guide specification.

ATC-6-1: The report, *Proceedings of a Workshop on Earthquake Resistance of Highway Bridges*, was published under a grant from NSF. Available through the ATC office. (Published 1979, 625 pages)

ABSTRACT: The report includes 23 state-of-the-art and state-of-practice papers on earthquake resistance of highway bridges. Seven of the twenty-three papers were authored by participants from Japan, New Zealand and Portugal. The Proceedings also contain recommendations for future research that were developed by the 45 workshop participants.

ATC-6-2: The report, *Seismic Retrofitting Guidelines for Highway Bridges*, was published under a contract with FHWA. Available through the ATC office. (Published 1983, 220 pages)

ABSTRACT: The Guidelines are the recommendations of a team of thirteen nationally recognized experts that included consulting engineers, academics, state highway engineers, and federal agency representatives. The Guidelines, applicable for use in all parts of the United States, include a preliminary screening procedure, methods for evaluating an existing bridge in detail, and potential retrofitting measures for the most common seismic deficiencies. Also included are special design requirements for various retrofitting measures.

ATC-7: The report, *Guidelines for the Design of Horizontal Wood Diaphragms*, was published under a grant from NSF. Available through the ATC office. (Published 1981, 190 pages)

ABSTRACT: Guidelines are presented for designing roof and floor systems so these can function as horizontal diaphragms in a lateral force resisting system. Analytical procedures, connection details and design examples are included in the Guidelines.

ATC-7-1: The report, *Proceedings of a Workshop of Design of Horizontal Wood Diaphragms*, was published under a grant from NSF. Available through the ATC office. (Published 1980, 302 pages)

ABSTRACT: The report includes seven papers on state-of-the-practice and two papers on recent research. Also included are recommendations for future research that were developed by the 35 workshop participants.

ATC-8: This report, *Proceedings of a Workshop on the Design of Prefabricated Concrete Buildings for Earthquake Loads*, was funded by NSF. Available through the ATC office. (Published 1981, 400 pages)

ABSTRACT: The report includes eighteen state-of-the-art papers and six summary papers. Also included are recommendations for future research that were developed by the 43 workshop participants.

ATC-9: The report, *An Evaluation of the Imperial County Services Building Earthquake Response and Associated Damage*, was published under a grant from NSF. Available through the ATC office. (Published 1984, 231 pages)

ABSTRACT: The report presents the results of an in-depth evaluation of the Imperial County Services Building, a 6-story reinforced concrete frame and shear wall building severely damaged by the October 15, 1979 Imperial Valley, California, earthquake. The report contains a review and evaluation of earthquake damage to the building; a review and evaluation of the seismic design; a comparison of the requirements of various building codes as they relate to the building; and conclusions and recommendations pertaining to future building code provisions and future research needs.

ATC-10: This report, *An Investigation of the Correlation Between Earthquake Ground Motion and Building Performance*, was funded by the U.S. Geological Survey (USGS). Available through the ATC office. (Published 1982, 114 pages)

ABSTRACT: The report contains an in-depth analytical evaluation of the ultimate or limit capacity of selected representative building framing types, a discussion of the factors affecting the seismic performance of buildings, and a summary and comparison of seismic design and seismic risk parameters currently in widespread use.

ATC-10-1: This report, *Critical Aspects of Earthquake Ground Motion and Building Damage Potential*, was co-funded by the USGS and the NSF. Available through the ATC office. (Published 1984, 259 pages)

ABSTRACT: This document contains 19 state-of-the-art papers on ground motion, structural response, and structural design issues presented by prominent engineers and earth scientists in an ATC seminar. The main theme of the papers is to identify the critical aspects of ground motion and building performance that currently are not being considered in building design. The report also contains conclusions and recommendations of working groups convened after the Seminar.

ATC-11: The report, *Seismic Resistance of Reinforced Concrete Shear Walls and Frame Joints: Implications of Recent Research for Design Engineers*, was published under a grant from NSF. Available through the ATC office. (Published 1983, 184 pages)

ABSTRACT: This document presents the results of an in-depth review and synthesis of research reports pertaining to cyclic loading of reinforced concrete shear walls and cyclic loading of joint reinforced concrete frames. More than 125 research reports published since 1971 are reviewed and evaluated in this report. The preparation of the report included a consensus process involving numerous experienced design professionals from throughout the United States. The report contains reviews of current and past design practices, summaries of research developments, and in-depth discussions of design implications of recent research results.

ATC-12: This report, *Comparison of United States and New Zealand Seismic Design Practices for Highway Bridges*, was published under a grant from NSF. Available through the ATC office. (Published 1982, 270 pages)

ABSTRACT: The report contains summaries of all aspects and innovative design procedures used in New Zealand as well as comparison of United States and New Zealand design practice. Also included are research recommendations developed at a 3-day workshop in New Zealand attended by 16 U.S. and 35 New Zealand bridge design engineers and researchers.

ATC-12-1: This report, *Proceedings of Second Joint U.S.-New Zealand Workshop on Seismic Resistance of Highway Bridges*, was published under a grant from NSF. Available through the ATC office. (Published 1986, 272 pages)

ABSTRACT: This report contains written versions of the papers presented at this 1985 Workshop as well as a list and prioritization of workshop recommendations. Included are summaries of research projects being conducted in both countries as well as state-of-the-practice papers on various aspects of design practice. Topics discussed include bridge design philosophy and loadings; design of columns, footings, piles, abutments and retaining structures; geotechnical aspects of foundation design; seismic analysis techniques; seismic retrofitting; case studies using base isolation; strong-motion data acquisition and interpretation; and testing of bridge components and bridge systems.

ATC-13: The report, *Earthquake Damage Evaluation Data for California*, was developed under a contract with the Federal Emergency Management Agency (FEMA). Available through the ATC office. (Published 1985, 492 pages)

ABSTRACT: This report presents expert-opinion earthquake damage and loss estimates for industrial, commercial, residential, utility and transportation facilities in California. Included are damage probability matrices for 78 classes of structures and estimates of time required to restore damaged facilities to pre-earthquake usability. The report also describes the inventory information essential for estimating economic losses and the methodology used to develop loss estimates on a regional basis.

ATC-14: The report, *Evaluating the Seismic Resistance of Existing Buildings*, was developed under a grant from the NSF. Available through the ATC office. (Published 1987, 370 pages)

ABSTRACT: This report, written for practicing structural engineers, describes a methodology for performing preliminary and detailed building seismic evaluations. The report contains a state-of-practice review; seismic loading criteria; data collection procedures; a detailed description of the building classification system; preliminary and detailed analysis procedures; and example case studies, including nonstructural considerations.

ATC-15: The report, *Comparison of Seismic Design Practices in the United States and Japan*, was published under a grant from NSF. Available through the ATC office. (Published 1984, 317 pages)

ABSTRACT: The report contains detailed technical papers describing design practices in the United States and Japan as well as recommendations emanating from a joint U.S.-Japan workshop held in Hawaii in March, 1984. Included are detailed descriptions of new seismic design methods for buildings in Japan and case studies of the design of specific buildings (in both countries). The report also contains an overview of the history and objectives of the Japan Structural Consultants Association.

ATC-15-1: The report, *Proceedings of Second U.S.-Japan Workshop on Improvement of Building Seismic Design and Construction Practices*, was published under a grant from NSF. Available through the ATC office. (Published 1987, 412 pages)

ABSTRACT: This report contains 23 technical papers presented at this San Francisco workshop in August, 1986, by practitioners and researchers from the U.S. and Japan. Included are state-of-the-practice papers and case studies of actual building designs and information on regulatory, contractual, and licensing issues.

ATC-15-2: The report, *Proceedings of Third U.S.-Japan Workshop on Improvement of Building Structural Design and Construction Practices*, was published jointly by ATC and the Japan Structural Consultants Association. Available through the ATC office. (Published 1989, 358 pages)

ABSTRACT: This report contains 21 technical papers presented at this Tokyo, Japan, workshop in July, 1988, by practitioners and researchers from the U.S., Japan, China, and New Zealand. Included are state-of-the-practice papers on various topics, including braced steel frame buildings, beam-column joints in reinforced concrete buildings, summaries of comparative U. S. and Japanese design, and base isolation and passive energy dissipation devices.

ATC-15-3: The report, *Proceedings of Fourth U.S.-Japan Workshop on Improvement of Building Structural Design and Construction Practices*, was published jointly by ATC and the Japan Structural Consultants Association. Available through the ATC office. (Published 1992, 484 pages)

ABSTRACT: This report contains 22 technical papers presented at this Kailua-Kona, Hawaii, workshop in August, 1990, by practitioners and researchers from the United States, Japan, and Peru. Included are papers on postearthquake building damage assessment; acceptable earth-quake damage; repair and retrofit of earthquake damaged buildings; base-isolated buildings, including Architectural Institute of Japan recommendations for design; active damping systems; wind-resistant design; and summaries of working group conclusions and recommendations.

ATC-15-4: The report, *Proceedings of Fifth U.S.-Japan Workshop on Improvement of Building Structural Design and Construction Practices*, was published jointly by ATC and the Japan Structural Consultants Association. Available through the ATC office. (Published 1994, 360 pages)

ABSTRACT: This report contains 20 technical papers presented at this San Diego, California workshop in September, 1992. Included are papers on performance goals/acceptable damage in seismic design; seismic design procedures and case studies; construction influences on design; seismic isolation and passive energy dissipation; design of irregular structures; seismic evaluation, repair and upgrading; quality control for design and construction; and summaries of working group discussions and recommendations.

ATC-16: This project, Development of a 5-Year Plan for Reducing the Earthquake Hazards Posed by Existing Nonfederal Buildings, was funded by FEMA and was conducted by a joint venture of ATC, the Building Seismic Safety Council and the Earthquake Engineering

Research Institute. The project involved a workshop in Phoenix, Arizona, where approximately 50 earthquake specialists met to identify the major tasks and goals for reducing the earthquake hazards posed by existing non-federal buildings nationwide. The plan was developed on the basis of nine issue papers presented at the workshop and workshop working group discussions. The Workshop Proceedings and Five-Year Plan are available through the Federal Emergency Management Agency, 500 "C" Street, S.W., Washington, DC 20472.

ATC-17: This report, *Proceedings of a Seminar and Workshop on Base Isolation and Passive Energy Dissipation*, was published under a grant from NSF. Available through the ATC office. (Published 1986, 478 pages)

ABSTRACT: The report contains 42 papers describing the state-of-the-art and state-of-the-practice in base-isolation and passive energy-dissipation technology. Included are papers describing case studies in the United States, applications and developments worldwide, recent innovations in technology development, and structural and ground motion issues. Also included is a proposed 5-year research agenda that addresses the following specific issues: (1) strong ground motion; (2) design criteria; (3) materials, quality control, and long-term reliability; (4) life cycle cost methodology; and (5) system response.

ATC-17-1: This report, *Proceedings of a Seminar on Seismic Isolation, Passive Energy Dissipation and Active Control,* was published under a grant from NSF. Available through the ATC office. (Published 1993, 841 pages)

ABSTRACT: The 2-volume report documents 70 technical papers presented during a two-day seminar in San Francisco in early 1993. Included are invited theme papers and competitively selected papers on issues related to seismic isolation systems, passive energy dissipation systems, active control systems and hybrid systems.

ATC-18: The report, *Seismic Design Criteria for Bridges and Other Highway Structures: Current and Future*, was published under a contract from the Multidisciplinary Center for Earthquake Engineering Research (formerly NCEER), with funding from the Federal Highway Administration. Available through the ATC office. (Published 1997, 152 pages)

ABSTRACT: This report documents the findings of a 4-year project to review and assess current seismic design criteria for new highway construction. The report addresses performance criteria, importance classification, definitions of seismic hazard for areas where damaging earthquakes have longer return periods, design ground motion, duration effects, site effects, structural response modification factors, ductility demand, design procedures, foundation and abutment modeling, soil-structure interaction, seat widths, joint details and detailing reinforced concrete for limited ductility in areas with low-to-moderate seismic activity. The report also provides lengthy discussion on future directions for code development and recommended research and development topics.

ATC-19: The report, *Structural Response Modification Factors* was funded by NSF and NCEER. Available through the ATC office. (Published 1995, 70 pages)

ABSTRACT: This report addresses structural response modification factors (R factors), which are used to reduce the seismic forces associated with elastic response to obtain design forces. The report documents the basis for current R values, how R factors are used for seismic design in other countries, a rational means for decomposing R into key components, a framework (and methods) for evaluating the key components of R, and the research necessary to improve the reliability of engineered construction designed using R factors.

ATC-20: The report, *Procedures for Postearthquake Safety Evaluation of Buildings*, was developed under a contract from the California Office of Emergency Services (OES), California Office of Statewide Health Planning and Development (OSHPD) and FEMA. Available through the ATC office (Published 1989, 152 pages)

ABSTRACT: This report provides procedures and guidelines for making on-the-spot evaluations and decisions regarding continued use and occupancy of earthquake damaged buildings. Written specifically for volunteer structural engineers and building inspectors, the report includes rapid and detailed evaluation procedures for inspecting buildings and posting them as "inspected" (apparently safe), "limited entry" or "unsafe". Also included are special procedures for evaluation of essential buildings (e.g., hospitals), and evaluation procedures for nonstructural elements, and geotechnical hazards.

ATC-20-1: The report, *Field Manual: Postearthquake Safety Evaluation of Buildings*, was developed under a contract from OES and OSHPD. Available through the ATC office (Published 1989, 114 pages)

ABSTRACT: This report, a companion Field Manual for the ATC-20 report, summarizes the postearthquake safety evaluation procedures in brief concise format designed for ease of use in the field.

ATC-20-2: The report, *Addendum to the ATC-20 Postearthquake Building Safety Procedures* was published under a grant from the NSF and funded by the USGS. Available through the ATC office. (Published 1995, 94 pages)

ABSTRACT: This report provides updated assessment forms, placards, and procedures that are based on an in-depth review and evaluation of the widespread application of the ATC-20 procedures following five earthquakes occurring since the initial release of the ATC-20 report in 1989.

ATC-20-3: The report, *Case Studies in Rapid Postearthquake Safety Evaluation of Buildings,* was funded by ATC and R. P. Gallagher Associates. Available through the ATC office. (Published 1996, 295 pages)

ABSTRACT: This report contains 53 case studies using the ATC-20 Rapid Evaluation procedure. Each case study is illustrated with photos and describes how a building was inspected and evaluated for life safety, and includes a completed safety assessment form and placard. The report is intended to be used as a training and reference manual for building officials, building inspectors, civil and structural engineers, architects, disaster workers, and others who may be asked to perform safety evaluations after an earthquake.

ATC-20-T: The report, *Postearthquake Safety Evaluation of Buildings Training Manual* was developed under a contract with FEMA. Available through the ATC office. (Published 1993, 177 pages; 160 slides)

ABSTRACT: This training manual is intended to facilitate the presentation of the contents of the ATC-20 and ATC-20-1. The training materials consist of 160 slides of photographs, schematic drawings and textual information and a companion training presentation narrative coordinated with the slides. Topics covered include: posting system; evaluation procedures; structural basics; wood frame, masonry, concrete, and steel frame structures; nonstructural elements; geotechnical hazards; hazardous materials; and field safety.

ATC-21: The report, *Rapid Visual Screening of Buildings for Potential Seismic Hazards: A Handbook*, was developed under a contract from FEMA. Available through the ATC office. (Published 1988, 185 pages)

ABSTRACT: This report describes a rapid visual screening procedure for identifying those buildings that might pose serious risk of loss of life and injury, or of severe curtailment of community services, in case of a damaging earthquake. The screening procedure utilizes a methodology based on a "sidewalk survey" approach that involves identification of the primary structural load resisting system and building materials, and assignment of a basic structural hazards score and performance modification factors based on observed building characteristics. Application of the methodology identifies those buildings that are potentially hazardous and should be analyzed in more detail by a professional engineer experienced in seismic design.

ATC-21-1: The report, *Rapid Visual Screening of Buildings for Potential Seismic Hazards: Supporting Documentation*, was developed under a contract from FEMA. Available through the ATC office. (Published 1988, 137 pages)

ABSTRACT: Included in this report are (1) a review and evaluation of existing procedures; (2) a listing of attributes considered ideal for a rapid visual screening procedure; and (3) a technical discussion of the recommended rapid visual screening procedure that is documented in the ATC-21 report.

ATC-21-2: The report, *Earthquake Damaged Buildings: An Overview of Heavy Debris and Victim Extrication*, was developed under a contract from FEMA. (Published 1988, 95 pages)

ABSTRACT: Included in this report, a companion volume to the ATC-21 and ATC-21-1 reports, is state-of-the-art information on (1) the identification of those buildings that might collapse and trap victims in debris or generate debris of such a size that its handling would require special or heavy lifting equipment; (2) guidance in identifying these types of buildings, on the basis of their major exterior features, and (3) the types and life capacities of equipment required to remove the heavy portion of the debris that might result from the collapse of such buildings.

ATC-21-T: The report, *Rapid Visual Screening of Buildings for Potential Seismic Hazards Training Manual* was developed under a contract with FEMA. Available through the ATC office. (Published 1996, 135 pages; 120 slides)

ABSTRACT: This training manual is intended to facilitate the presentation of the contents of the ATC-21 report. The training materials consist of 120 slides and a companion training presentation narrative coordinated with the slides. Topics covered include: description of procedure, building behavior, building types, building scores, occupancy and falling hazards, and implementation.

ATC-22: The report, *A Handbook for Seismic Evaluation of Existing Buildings (Preliminary)*, was developed under a contract from FEMA. Available through the ATC office. (Originally published in 1989; revised by BSSC and published as the *NEHRP Handbook for Seismic Evaluation of Existing Buildings* in 1992, 211 pages)

ABSTRACT: This handbook provides a methodology for seismic evaluation of existing buildings of different types and occupancies in areas of different seismicity throughout the United States. The methodology, which has been field tested in several programs nationwide, utilizes the information and procedures developed for and documented in the ATC-14 report. The handbook includes checklists, diagrams, and sketches designed to assist the user.

ATC-22-1: The report, *Seismic Evaluation of Existing Buildings: Supporting Documentation*, was developed under a contract from FEMA. (Published 1989, 160 pages)

ABSTRACT: Included in this report, a companion volume to the ATC-22 report, are (1) a review and evaluation of existing buildings seismic evaluation methodologies; (2) results from field tests of the ATC-14 methodology; and (3) summaries of evaluations of ATC-14 conducted by the National Center for Earthquake Engineering Research (State University of New York at Buffalo) and the City of San Francisco.

ATC-23A: The report, *General Acute Care Hospital Earthquake Survivability Inventory for California, Part A: Survey Description, Summary of Results, Data Analysis and Interpretation*, was developed under a contract from the Office of Statewide Health Planning and Development (OSHPD), State of California. Available through the ATC office. (Published 1991, 58 pages)

ABSTRACT: This report summarizes results from a seismic survey of 490 California acute care hospitals. Included are a description of the survey procedures and data collected, a summary of the data, and an illustrative discussion of data analysis and interpretation that has been provided to demonstrate potential applications of the ATC-23 database.

ATC-23B: The report, *General Acute Care Hospital Earthquake Survivability Inventory for California, Part B: Raw Data*, is a companion document to the ATC-23A Report and was developed under the above-mentioned contract from OSHPD. Available through the ATC office. (Published 1991, 377 pages)

ABSTRACT: Included in this report are tabulations of raw general site and building data for 490 acute care hospitals in California.

ATC-24: The report, *Guidelines for Seismic Testing of Components of Steel Structures*, was jointly funded by the American Iron and Steel Institute (AISI), American Institute of Steel Construction (AISC), National Center for Earthquake Engineering Research (NCEER), and NSF. Available through the ATC office. (Published 1992, 57 pages)

ABSTRACT: This report provides guidance for most cyclic experiments on components of steel structures for the purpose of consistency in experimental procedures. The report contains recommendations and companion commentary pertaining to loading histories, presentation of test results, and other aspects of experimentation. The recommendations are written specifically for experiments with slow cyclic load application.

ATC-25: The report, *Seismic Vulnerability and Impact of Disruption of Lifelines in the Conterminous United States*, was developed under a contract from FEMA. Available through the ATC office. (Published 1991, 440 pages)

ABSTRACT: Documented in this report is a national overview of lifeline seismic vulnerability and impact of disruption. Lifelines considered include electric systems, water systems, transportation systems, gas and liquid fuel supply systems, and emergency service facilities (hospitals, fire and police stations). Vulnerability estimates and impacts developed are presented in terms of estimated first approximation direct damage losses and indirect economic losses.

ATC-25-1: The report, *A Model Methodology for Assessment of Seismic Vulnerability and Impact of Disruption of Water Supply Systems*, was developed under a contract from FEMA. Available through the ATC office. (Published 1992, 147 pages)

ABSTRACT: This report contains a practical methodology for the detailed assessment of seismic vulnerability and impact of disruption of water supply systems. The methodology has been designed for use by water system operators. Application of the methodology enables the user to develop estimates of direct damage to system components and the time required to restore damaged facilities to pre-earthquake usability. Suggested measures for mitigation of seismic hazards are also provided.

ATC-28: The report, *Development of Recommended Guidelines for Seismic Strengthening of Existing Buildings, Phase I: Issues Identification and Resolution*, was developed under a contract with FEMA. Available through the ATC office. (Published 1992, 150 pages)

ABSTRACT: This report identifies and provides resolutions for issues that will affect the development of guidelines for the seismic strengthening of existing buildings. Issues addressed include: implementation and format, coordination with other efforts, legal and political, social, economic, historic buildings, research and technology, seismicity and mapping, engineering philosophy and goals, issues related to the development of specific provisions, and nonstructural element issues.

ATC-29: The report, *Proceedings of a Seminar and Workshop on Seismic Design and Performance of Equipment and Nonstructural Elements in Buildings and Industrial Structures*, was developed under a grant from NCEER and NSF. Available through the ATC office. (Published 1992, 470 pages)

ABSTRACT: These Proceedings contain 35 papers describing state-of-the-art technical information pertaining to the seismic design and performance of equipment and nonstructural elements in buildings and industrial structures. The papers were presented at a seminar in Irvine, California in 1990. Included are papers describing current practice, codes and regulations; earthquake performance; analytical and experimental investigations; development of new seismic qualification methods; and research, practice, and code development needs for specific elements and systems. The report also includes a summary of a proposed 5-year research agenda for NCEER.

ATC-29-1: The report, *Proceedings Of Seminar On Seismic Design, Retrofit, And Performance Of Nonstructural Components*, was developed under a grant from NCEER and NSF. Available through the ATC office. (Published 1998, 518 pages)

ABSTRACT: These Proceedings contain 38 papers presenting current research, practice, and informed thinking pertinent to seismic design, retrofit, and performance of nonstructural components. The papers were presented at a seminar in San Francisco, California, in 1998. Included are papers describing observed performance in recent earthquakes; seismic design codes, standards, and procedures for commercial and institutional buildings; seismic design issues relating to industrial and hazardous material facilities; design, analysis, and testing; and seismic evaluation and rehabilitation of conventional and essential facilities, including hospitals.

ATC-30: The report, *Proceedings of Workshop for Utilization of Research on Engineering and Socioeconomic Aspects of 1985 Chile and Mexico Earthquakes*, was developed under a grant from the NSF. Available through the ATC office. (Published 1991, 113 pages)

ABSTRACT: This report documents the findings of a 1990 technology transfer workshop in San Diego, California, co-sponsored by ATC and the Earthquake Engineering Research Institute. Included in the report are invited papers and working group recommendations on geotechnical issues, structural response issues, architectural and urban design considerations, emergency response planning, search and rescue, and reconstruction policy issues.

ATC-31: The report, *Evaluation of the Performance of Seismically Retrofitted Buildings*, was developed under a contract from the National Institute of Standards and Technology (NIST, formerly NBS) and funded by the USGS. Available through the ATC office. (Published 1992, 75 pages)

ABSTRACT: This report summarizes the results from an investigation of the effectiveness of 229 seismically retrofitted buildings, primarily unreinforced masonry and concrete tilt-up buildings. All buildings were located in the areas affected by the 1987 Whittier Narrows, California, and 1989 Loma Prieta, California, earthquakes.

ATC-32: The report, *Improved Seismic Design Criteria for California Bridges: Provisional Recommendations*, was funded by the California Department of Transportation (Caltrans). Available through the ATC office. (Published 1996, 215 Pages)

ABSTRACT: This report provides recommended revisions to the current *Caltrans Bridge Design Specifications* (BDS) pertaining to seismic loading, structural response analysis, and component design. Special attention is given to design issues related to reinforced concrete components, steel components, foundations, and conventional bearings. The recommendations are based on recent research in the field of bridge seismic design and the performance of Caltrans-designed bridges in the 1989 Loma Prieta and other recent California earthquakes.

ATC-34: The report, *A Critical Review of Current Approaches to Earthquake Resistant Design*, was developed under a grant from NCEER and NSF. Available through the ATC office. (Published, 1995, 94 pages)

ABSTRACT. This report documents the history of U. S. codes and standards of practice, focusing primarily on the strengths and deficiencies of current code approaches. Issues addressed include: seismic hazard analysis, earthquake collateral hazards, performance objectives, redundancy and configuration, response modification factors (R factors), simplified analysis procedures, modeling of structural components, foundation design, nonstructural component design, and risk and reliability. The report also identifies goals that a new seismic code should achieve.

ATC-35: This report, *Enhancing the Transfer of U.S. Geological Survey Research Results into Engineering Practice* was developed under a contract with the USGS. Available through the ATC office. (Published 1996, 120 pages)

ABSTRACT: The report provides a program of recommended "technology transfer" activities for the USGS; included are recommendations pertaining to management actions, communications with practicing engineers, and research activities to enhance development and transfer of information that is vital to engineering practice.

ATC-35-1: The report, *Proceedings of Seminar on New Developments in Earthquake Ground Motion Estimation and Implications for Engineering Design Practice*, was developed under a cooperative agreement with USGS. Available through the ATC office. (Published 1994, 478 pages)

ABSTRACT: These Proceedings contain 22 technical papers describing state-of-the-art information on regional earthquake risk (focused on five specific regions--California, Pacific Northwest, Central United States, and northeastern North America); new techniques for estimating strong ground motions as a function of earthquake source, travel path, and site parameters; and new developments specifically applicable to geotechnical engineering and the seismic design of buildings and bridges.

ATC-37: The report, *Review of Seismic Research Results on Existing Buildings*, was developed in conjunction with the Structural Engineers Association of California and California Universities for Research in Earthquake Engineering under a contract from the California Seismic Safety Commission (SSC). Available through the Seismic Safety Commission as Report SSC 94-03. (Published, 1994, 492 pages)

ABSTRACT. This report describes the state of knowledge of the earthquake performance of nonductile concrete frame, shear wall, and infilled buildings. Included are summaries of 90 recent research efforts with key results and conclusions in a simple, easy-to-access format written for practicing design professionals.

ATC-40: The report, *Seismic Evaluation and Retrofit of Concrete Buildings*, was developed under a contract from the California Seismic Safety Commission. Available through the ATC office. (Published, 1996, 612 pages)

ABSTRACT. This 2-volume report provides a state-of-the-art methodology for the seismic evaluation and retrofit of concrete buildings. Specific guidance is provided on the following topics: performance objectives; seismic hazard; determination of deficiencies; retrofit strategies; quality assurance procedures; nonlinear static analysis procedures; modeling rules; foundation effects; response limits; and nonstructural components. In 1997 this report received the Western States Seismic Policy Council "Overall Excellence and New Technology Award."

ATC-44: The report, *Hurricane Fran, South Carolina, September 5, 1996: Reconnaissance Report*, is available through the ATC office. (Published 1997, 36 pages.)

ABSTRACT: This report represents ATC's expanded mandate into structural engineering problems arising from wind storms and coastal flooding. It contains information on the causative hurricane; coastal impacts, including storm surge, waves, structural forces and erosion; building codes; observations and interpretations of damage; and lifeline performance. Conclusions address man-made beach nourishment, the effects of missile-like debris, breaches in the sandy barrier islands, and the timing and duration of such investigations.

ATC-R-1: The report, *Cyclic Testing of Narrow Plywood Shear Walls,* was developed with funding from the Henry J. Degenkolb Memorial Endowment Fund of the Applied Technology Council. Available through the ATC office (Published 1995, 64 pages)

ABSTRACT: This report documents ATC's first self-directed research program: a series of static and dynamic tests of narrow plywood wall panels having the standard 3.5-to-1 height-to-width ratio and anchored to the sill plate using typical bolted, 9-inch, 5000-lb. capacity hold-down devices. The report provides a description of the testing program and a summary of results, including comparisons of drift ratios found during testing with those specified in the seismic provisions of the 1991 *Uniform Building Code*.

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